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# Deterioration of Bridge Decks and Review of the WSDOT Bridge Deck Program

## WA-RD 55.1

Final

September 1982

**Washington State Department of Transportation**
Public Transportation and Planning
In Cooperation with
United States Department of Transportation
Federal Highway Administration

TECHNICAL REPORT STANDARD TITLE PAGE

| | | |
|---|---|---|
| 1. Report No.<br>WA-RD-55.1 | 2. Government Accession No. | 3. Recipient's Catalog No. |

| | |
|---|---|
| 4. Title and Subtitle<br>Deterioration of Concrete Bridge Decks and Review of the WSDOT Bridge Deck Program | 5. Report Date<br>September, 1982<br>6. Performing Organization Code |
| 7. Author(s)<br>Khossrow Babaei and Ronald L. Terrel | 8. Performing Organization Report No.<br>TRAC 82-1 |
| 9. Performing Organization Name and Address<br>Washington State Transportation Center (TRAC)<br>121 More Hall (FX-10)<br>University of Washington<br>Seattle, WA 98195 | 10. Work Unit No.<br>11. Contract or Grant No.<br>Y-2463 |
| 12. Sponsoring Agency Name and Address<br>Washington State Department of Transportation<br>Highway Administration Building, KF-01<br>Olympia, WA 98504 | 13. Type of Report and Period Covered<br>Final Report<br>4/15/82 - 9/30/82<br>14. Sponsoring Agency Code |

15. Supplementary Notes

Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, study title "Evaluation of WSDOT Concrete Bridge Deck Program"

16. Abstract

This report reviews the program developed by WSDOT to minimize premature deterioration of concrete bridge decks subject to chloride deicing chemicals. The review includes WSDOT's deck rehabilitation strategy, deck protective systems, deck evaluation techniques, and research and special investigations. The rport includes a number of recommendations based on information obtained from previous research relative to the WSDOT program.

Among protective systems used or proposed for use, cathodic protection appeared to be the most promising approach, short of removal of all chloride-contaminated concrete, that will stop active corrosion of steel in bridge decks. Information evolving from the study also pointed to waterproofing membrane and AC overlay as a relatively inexpensive approach to protect existing bridge decks against corrosion. However, there appears to be a lack of hard data to support the effectiveness of this system.

It is recommended that half-cell corrosion detection be included in the evaluation of bridge decks in combination with other techniques such as chloride analysis and chain dragging delamination detection.

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|---|---|
| 17. Key Words<br>reinforced concrete, bridge deck, steel corrosion, deicing chlorides, deterioration | 18. Distribution Statement |

| | | | |
|---|---|---|---|
| 19. Security Classif. (of this report)<br>unclassified | 20. Security Classif. (of this page)<br>unclassified | 21. No. of Pages<br>123 | 22. Price |

Form DOT F 1700.7 (8-69)
HWY FORM 341-004 6/72

# DETERIORATION OF CONCRETE BRIDGE DECKS AND REVIEW OF THE WSDOT BRIDGE DECK PROGRAM

Report No. TRAC 82-1

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September, 1982

# ACKNOWLEDGMENT

This work was sponsored by the Washington State Department of Transportation, in cooperation with the Federal Highway Administration, and was conducted by the Washington State Transportation Center at the University of Washington.

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# SUMMARY

The objective of this study of the WSDOT Bridge Deck Program was to evaluate the program developed and proposed by the Washington State Department of Transportation in order to minimize the premature deterioration of reinforced concrete bridge decks caused by using chloride deicing chemicals. The WSDOT Bridge Deck Program, which is in agreement with FHPM 6-7-2-7, "Concrete Bridges," dated April 5, 1976, and FHWA "Region 10 Position Paper on Concrete Bridge Decks," dated August 3, 1981, was prepared to establish guidelines for the protection of uncontaminated bridge decks and reconstruction of those which are contaminated or in advancing stages of deterioration.

A literature survey was made to obtain information developed from previous research relative to the WSDOT program. Research efforts concerning the causes of concrete bridge deck deterioration and current practices relating to the methods of field appraisal, as well as deck protective systems, were reviewed using available literature. The WSDOT program was reviewed and compared to this available information. This included WSDOT's deck rehabilitation strategy, deck protective systems, deck evaluation techniques, and research and special investigations. The scope of this study did not permit review of WSDOT's budgeting or economic analysis of the bridge decks program.

The following is a summary of recommendations made during this study regarding the WSDOT program that have received general support from the advisory panel established to guide the project.

## Bridge Deck Protection and Rehabilitation Strategy

It is believed that WSDOT's strategy to prevent chloride contamination of existing uncontaminated bridge decks could prevent future high cost rehabilitation and reconstruction of these structures. It is also suggested that use of possible alternatives to replace or minimize salting in the future should be investigated.

The WSDOT program generally offers two alternatives for rehabilitation of contaminated bridge decks: "permanent" and "experimental cost-effective" reconstruction. However, the program does not include criteria for reconstruction and installation of a protective system at different stages in the life of a bridge deck. It appears that WSDOT may need to develop a systematic approach

to evaluate and rehabilitate bridge decks that would include appropriate criteria. One possibility would be to organize the procedure into a "Bridge Deck Management System: (BDMS) patterned after the successful Pavement Management System (PMS) already developed in Washington by WSDOT. A suitably designed BDMS (or Bridge Management System, if other bridge features were included) would enable the engineer to use collected data on the condition of bridge decks, along with available criteria, to not only evaluate the present condition of the bridge, but after some experience and history, to begin to predict the rate of deterioration. This information would be a strong tool for management to make rational decisions about expenditure of available funds. A very important aspect of this system is the feedback on how well both new and rehabilitated bridges are performing. Without this feedback, meaningful criteria would be difficult if not impossible to develop.

A well-designed BDMS, for example could provide for alternatives based on cost-effective methods. The alternative of experimental cost-effective reconstruction as defined in the WSDOT Bridge Deck Program which does not call for removal of all chloride-contaminated concrete may be acceptable, but only if it can be shown that permanent reconstruction is not economically justified. For example, the extent of contaminated concrete is usually not known in sufficient detail prior to reconstruction to enable lump sum bidding. Consequently, the actual cost of a project based on cost plus procedure that requires complete removal of contaminated concrete may far exceed the savings in terms of extended bridge life. Depending upon other factors, the economical choice may not be permanent reconstruction.

## Deck Protective Systems

Epoxy coating the top mat of reinforcing steel, which is specified by WSDOT for new bridges, is in fact a practical and inexpensive solution for protection of new bridge deck installations against corrosion. However, the system has the potential of developing an accelerated corrosion pattern of cracks, holes, and damaged areas on the coating caused mainly by fabrication and handling of the coated bars. Repetitive loading could also cause a wearing away of the coating. Therefore, performance of this system should be watched closely by WSDOT. If the preponderance of data shows good performance, the system can be considered an efficient solution to prevent deterioration of bridge decks. Epoxy coating both top and bottom mats, on the other hand, could provide an added protection.

Dense concrete and latex-modified concrete overlays are specified by WSDOT for existing bridges. Regardless of some local bond failures and crackings that have been reported for these protective methods, they have shown generally good performance and are believed to be practical for protecting existing decks. These systems are well-suited to repair of badly spalled or scaled decks. However, they have a disadvantage in that when placed over existing cracks, they will quickly reflect those cracks.

Asphalt concrete overlays with waterproofing membranes are also currently used by WSDOT to protect existing bridges. This system is an inexpensive choice compared with the dense and latex-modified overlays, but there may be problems with potential movement of the membrane, cracking of the membrane by traffic loading, or cracking of the membrane from weathering, which may go undetected and cause concealed corrosion damage. There appears to be a lack of hard data to support the effectiveness of this system. Therefore, further evaluation of the system and factors affecting the durability of the membrane is suggested. Case histories of a few bridges having received waterproofing membrane and asphalt overlay should be studied carefully to evaluate the results.

WSDOT is considering future application of cathodic protection on existing bridge decks. Cathodic protection seems to be the only protective system, short of removal of all chloride-contaminated concrete, that will stop active corrosion in bridge decks. Cathodic protection devices, costs, and systems should be explored in detail and further evaluation of the system is recommended through installation in one pilot project. The FHWA demonstration project would appear to be an excellent opportunity to try it on at least one bridge.

## Field Evaluation Techniques

Currently only chloride analysis and chain dragging delamination detection are included in WSDOT's detailed field appraisal of concrete bridge decks. Chloride analysis is a valid method to define areas of contaminated concrete. However, it is not recommended to be used as the only procedure to evaluate concrete bridge decks. Other evaluation techniques, such as reinforcing steel corrosion detection (half-cell) and delamination detection (chain dragging), are nondestructive and may be more rapid and economical than chloride analysis.

While half-cell corrosion detection seems to have most potential for adaptation as a standard test to verify corrosion existence and its relative magnitude of extent, it has the disadvantage of operator error from poor connections, poor conductivity, etc. Its advantages include speed and reduced damage to a structure. It is recommended that investigations be made to refine this technique and improve procedures for its use as a routine evaluation tool.

It is also recommended that a pachometer survey be conducted in WSDOT's evaluation of bare concrete decks to determine if observed deterioration is related to insufficient cover over the reinforcing steel. The pachometer survey is also needed to locate the core samples for chloride analysis to avoid drilling through reinforcing bars.

# INTRODUCTION

The purpose of this investigation is to evaluate the program developed and proposed by the Washington State Department of Transportation in order to minimize the premature deterioration of reinforced concrete bridge decks caused by using chloride deicing chemicals. The WSDOT Bridge Deck Program, which is in agreement with FHPM 6-7-2-7, "Concrete Bridges," dated April 5, 1976 and FHWA "Region 10 Position Paper on Concrete Bridge Decks" (37), dated August 3, 1981, was prepared to establish guidelines for the protection of uncontaminated bridge decks and the reconstruction of those which are contaminated or in advancing stages of deterioration.

The primary study effort was directed toward obtaining information developed from previous research relative to the WSDOT program. Research efforts concerning the causes of concrete bridge deck deterioration and current practices relating to the methods of field appraisal, as well as deck protective systems, were reviewed using available literature. The review is presented in a state-of-the-art form in the report.

The WSDOT Bridge Deck Program is a document outlining the elements of the Bridge Branch's approach to evaluation and maintenance of bridge decks. Following the development of past experience, each element of the WSDOT program was reviewed and discussed in terms of how it fit into the state-of-the-art. The topics in the WSDOT report are: (a) deck rehabilitation strategy, (b) deck protective systems, (c) deck evaluation techniques, and (d) research and special investigations.

The scope of this study does not include a review of WSDOT's budgeting or economic analyses of bridge decks, since these items were not part of the program being reviewed. Finally, the study includes a number of recommendations regarding the WSDOT program which evolved from the project staff and discussion with WSDOT personnel, and they also have the support of the advisory panel established to guide the project.

A key part of this project was the critical review provided by outside advisors who were selected for their expertise and experience in the design, performance, evaluation and repair of concrete. Following a review of the available documents on the WSDOT Bridge Deck Program, the team met in Seattle for detailed discussion of various elements of the program and together they developed comments and suggestions which have been incorporated throughout this report. The authors of this report and TRAC would like to acknowledge the work of the advisory team, which consisted of the following:

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# INFORMATION SEARCH AND LITERATURE SURVEY

The primary effort was directed toward obtaining information developed from previous research relative to this study. This was done to prepare a state-of-the-art report to determine what was already known about the causes of concrete bridge deck deterioration, methods of evaluating concrete bridge decks and systems, materials and procedures that could control or offer promise of controlling corrosion of reinforcing steel in concrete bridge decks.

Research efforts concerning the causes of concrete bridge deck deterioration and current practices relating to the methods of field appraisal, as well as deck protective systems, were reviewed using available literature. The review is presented in Appendix A in the form of a state-of-the-art report, with pertinent references inserted when needed to indicate information sources.

The state-of-the-art report was prepared specifically for the bridge and materials engineer. Accordingly, the basic considerations of the corrosion process are initially discussed. This is followed by a review of the methods for evaluating existing bridge decks, including delamination detection, half-cell corrosion detection, measurement of concrete cover, and measurement of chloride content. Next, the discussion is about procedures to prevent the corrosion of reinforcing steel in new and existing concrete bridge decks, including epoxy coating reinforcing steel, corrosion inhibitors, low slump concrete overlay, polymer-modified concrete overlay, internally sealed concrete overlay, waterproofing membranes, and cathodic protection.

The results of the review of the published literature and other information obtained may be briefly summarized as follows:

## Corrosion of Reinforcing Steel

a. Chloride-free, bottom rebar mat is the major cathode in the corrosion of steel in concrete. The reason for this is that there is a large potential difference between rebars in the chloride-bearing area (top rebar mat) and rebars in chloride-free concrete. Removal of molecular oxygen at the anode would not stop the corrosion of steel and the corrosion can continue through iron chloride complexing and hydrolysis (7).

b. Field rehabilitation techniques which involve removal of only part of the chloride-contaminated concrete followed by patching with chloride-free concrete are probably self-defeating because large cathode sites are created and they may accelerate deterioration of the concrete adjacent to the patches. In this case the steel and concrete should be coated with a liquid epoxy prior to placing patch concrete or a dielectric repair material such as an epoxy mortar should be used (7, 1).

## Methods of Field Appraisal

a. Chloride analysis of bridge deck concrete as the sole evaluation technique to define the areas of concrete which must be removed prior to a permanent repair is not recommended. Other evaluation techniques (chain dragging delamination detection and half-cell corrosion detection) can define a portion of the concrete that must be removed, preventing the need to core these areas for chloride analysis (8). These techniques are non-destructive and could be more rapid and thus economical than chloride analysis.

b. Pachometers can be used to determine the clear concrete cover over the reinforcing steel as well as the location of the cores for chloride analysis. Rolling pachometers can be used effectively to expedite the process (1, 15).

c. The rapid chloride permeability test has a great value only in judging chloride permeability of different products and materials. Concretes can be tested nondestructively either in the laboratory or in the field.

## Bridge Deck Protective Systems

a. Epoxy films on reinforcing bars can withstand a moderate amount of abuse. Present fabrication and handling methods should be modified and extensively damaged areas should be repaired just prior to casting the concrete (18). A potential problem may exist when damaged epoxy-coated rebars in chloride-bearing concrete (top rebar mat) are electrically coupled to large quantities of uncoated steel in chloride-free concrete (bottom rebar mat). In this case a large cathode (bottom rebar mat) could develop and drive corrosion rapidly at damaged areas (7). The development of this corrosion appears to be a practical concern only when the total bare damaged area exceeds 0.25%. However, if both top and bottom mats are coated, the specifications allow up to 2% bare area (2). This is because the cathode surface area will be very small and limited only to the damaged areas of the bottom mat.

b. Active cracks on existing bridge decks generally preclude the use of rigid concrete overlays because they are very susceptible to reflection cracks (1). Stated differently, the existing crack is a stress raiser that causes that crack to continue to propagate, which may limit the overlay's service life.

c. Low slump concrete overlays use inexpensive materials but the placing operation requires the use of specialized finishing machines. Good quality control and inspection procedures are also required for the success of low slump concrete overlays (1).

d. Latex-modified concrete overlays utilize expensive material but are placed by conventional equipment. Superior performance of latex-modified concrete in chloride penetration tests have led to overlays being thinner than most low slump concrete overlays (1, 20).

Bridge Deck Protective Systems (continued)

e. Internally sealed concrete overlays are similar in handling and placing to conventional concrete. However, its economic feasibility remains doubtful using present heat treatment procedures. Research is continuing in an attempt to eliminate the heat treatment entirely.

f. Most of the waterproofing membrane systems in service are not performing their function in a completely satisfactory manner. The major reasons for this are (25):

   1. Permeability of membranes caused by pinholes and bubbles in the film.

   2. Improper construction practices, resulting in damage to the membrane during placement of asphalt concrete wearing course.

   3. Inability of some membrane systems to bridge moving cracks at low temperatures and to withstand softening in the wheelpaths at high temperatures.

g. Cathodic protection is the only protective system, short of removal of all chloride-contaminated concrete, that is sure to stop active corrosion in bridge decks (1). The success of cathodic protection depends on the electrical continuity between the various metallic members of the reinforcement. Recent studies indicate that the electrical continuity of the reinforcing steel in bridge decks may prove to be satisfactory. A necessary precaution when using conductive mixtures and overlays on the bridge deck is to ensure that the concrete of the bridge deck is sound and properly air-entrained. Care should also be taken to avoid possible overprotection of the structure. Recent development of non-overlay systems seems to be promising in which the entire cathodic protection system is placed in slots cut in the deck surface (5, 7, 28).

# WSDOT BRIDGE DECK PROGRAM

## I. INTRODUCTION

In order to minimize the deterioration of concrete bridge decks caused by application of chloride deicing chemicals in the State of Washington, WSDOT has prepared a program which is in agreement with FHPM 6-7-2-7, "Concrete Bridges," dated April 5, 1976 and FHWA "Region 10 Position Paper on Concrete Bridge Decks," dated August 3, 1981. The objective of the program is to establish guidelines for the restoration and protection of the bridges which are contaminated or in advancing stages of deterioration as well as the protection of uncontaminated bridge decks.

Program updates, however, are anticipated by WSDOT and may occur on a frequent basis due to the intensive research and advancing state-of-the-art currently being experienced.

This section of the report will briefly review and, wherever appropriate, discuss the WSDOT program. The review will include WSDOT's general deck rehabilitation strategy, current deck protective systems, current deck evaluation methods, and research and special investigations.

## II. DEFINITIONS (as used in this directive) (2)

### A. Reconstruction

"Reconstruction" is the restoration of the structural integrity of a bridge deck which may include partial restoration or complete removal and replacement of the existing deteriorated bridge deck. Partial restoration of an existing bridge deck includes the partial removal and replacement of all deteriorated and/or inadequate components.

### B. Protective System

"Protective system" denotes a system used to protect bridge decks from early deterioration due to reinforcing steel corrosion induced by highway deicing chemicals.

## III. EVALUATING EXISTING BRIDGE DECKS

### A. Routine Field Condition Survey

A field condition survey is needed to identify existing bridge decks that are possibly contaminated with deicing salts. The survey would also include an analysis of the extent of possible structural deficiencies. The information from the preliminary survey may be used to establish reconstruction priorities or to determine structural adequacy of the deck. The following are some examples of conditions recommended by FHWA and considered by WSDOT that can be readily determined or established, one or more of which may warrant reconstruction and/or protective systems (2):

(a) Maintenance records which indicate frequent application of deicing chemicals on this specific route.

(b) Visible concrete spalls which have occurred in the deck riding surface and/or evidence of unsound concrete in the bottom exposed surface of the deck slab which may indicate structural failure.

(c) Extensive deterioration of the asphaltic overlay logically due to underlying concrete deterioration.

(d) Evidence of delaminations (horizontal fracture planes) in the concrete deck.

(e) Evidence of reinforcing steel corrosion.

(f) Evidence of inadequate concrete cover over the reinforcing steel.

(g) Evidence of concrete bridge deck deterioration due to wind-blown salt water spray.

(h) When the structural adequacy of a bridge deck to carry current traffic loads is questioned, the field condition survey should also include a review and analysis of the extent of deficiencies and the feasibility of repair. Economics, traffic maintenance, etc., need to be evaluated when balancing the feasibility of structural restoration against complete replacement.

### B. Detailed Field Appraisal

Where the field condition survey has shown that reconstruction and/or protective systems may be warranted, a detailed field appraisal should be made to further define the inadequacies of the existing deck. Based on the information from the detailed field appraisal, an acceptable reconstruction procedure can be developed. The following, to the extent appropriate, are recommended by FHWA and considered by WSDOT as the components of an evaluation system (2):

(a) Delamination detection with appropriate equipment to determine extent of internal fractures in the concrete.

(b) Determination of the extent of reinforcing steel corrosion by the use of a half-cell corrosion detection device.

(c) Determination of areas with inadequate concrete cover over the reinforcing steel by the use of appropriate equipment.

(d) Chemical analysis to determine extent of chloride contamination.

## C. FHWA Category Classification of Existing Bridges

Based on the detailed field condition survey, three categories of condition as described below are recommended by FHWA and considered by WSDOT which can be used as guidelines. The limits will be established based on past experience and cost factors. If the categories overlap, judgment will be based on engineering, economic and other factors to properly categorize an existing bridge deck (2).

(a) Category 1 - Extensive Active Corrosion

Five per cent or more of the deck visibly spalled, OR:

Forty per cent or more of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of nonduplicating areas consisting of the following: (1) spalls, (2) delaminations, (3) electrical potentials over 0.35 volts (CSE) and (4) chloride content samples greater than 2 pounds of chloride per cubic yard (1.18 kg/$m^3$) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations and potentials more negative than -0.35 volts.

(b) Category 2 - Moderate Active Corrosion

Zero to 5 per cent of the deck visibly spalled, OR:

Five to 40 per cent of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of nonduplicating areas consisting of the following: (1) spalls, (2) delaminations, (3) electrical potentials over 0.35 volts (CSE) and (4) chloride content samples greater than 2 pounds of chloride per cubic yard (1.18 kg/$m^3$) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations and potentials more negative than -0.35 volts.

(c) Category 3 - Light to No Active Corrosion

No visible spalls, OR:

Zero to 5 per cent of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of non-duplicating areas consisting of the following: (1) delaminations, (2) electrical potentials over 0.35 volts (CSE), and (3) chloride content samples greater than 2 pounds chloride per cubic yard (1.18 kg/m$^3$) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations, and potentials more negative than -0.35 volts.

## IV. RECONSTRUCTION

In an effort to alleviate bridge deck deterioration and provide bridge decks with a reasonable service life, FHWA has established the following policy which is also considered by WSDOT to assure that bridge deck reconstruction will be as nearly permanent as judicious economics and current technology will permit (2).

### A. Reconstruction Work

Reconstruction includes all concrete deck restoration work required to assure satisfactory performance of the concrete deck and of the adopted protective system if required. This may include items such as the removal of existing overlays, removal and replacement of all deteriorated and/or severely contaminated concrete, removal and replacement of all deteriorated reinforcing steel or the complete removal and replacement of an entire bridge deck where necessary.

### B. Reconstruction Procedures

The following procedures are acceptable for use on Federal Aid projects:

1. Permanent Reconstruction

(a) Permanent restoration requires a structurally sound bridge deck and the removal of all delaminated concrete, highly chloride-contaminated concrete and also deteriorated concrete and rebars in areas of active corrosion. With proper restoration procedures and the installation of an approved protective system, this type of reconstruction is considered permanent and nonexperimental.

(b) Permanent reconstruction, however, will require a complete deck survey as to delaminations, corrosion potentials, and chloride contents, except where visual and delamination surveys indicate complete deck replacement as the obvious economical alternative.

2. Experimental Cost Effective Reconstruction

(a) This procedure is considered experimental, may be undertaken on concrete bridge decks found to be structurally sound, and requires restoration procedures sufficient to assure an estimated extended life for the deck of 10 to 15 years. This alternative allows salt contaminated concrete to be left in place, but the system of restoration must be considered effective based on previous successful experimental installations.

(b) Experimental consideration requires that the first three installations and 10 per cent of the remaining installations shall be identified as test structures and be completely surveyed prior to restoration. Ideally, these surveys should represent realistic sampling of the full range of deck conditions being restored.

(c) Immediately after restoration these test structures shall be resurveyed for corrosion potentials and, if membranes are used, resistivity readings shall be recorded. Periodically thereafter, on all test structures, corrosion potential levels, resistivity readings where appropriate, and the presence of delaminations where practical, shall be determined. Generally, these resurveys should not include chloride sampling unless low membrane resistivity test readings and/or increasing corrosion potentials or delaminations clearly indicate that they are needed. These periodic surveys should be performed at two- or three-year intervals depending on magnitude and frequency of salting applications unless otherwise agreed upon.

## V. ACCEPTABLE RESTORATION PROCEDURES

Based on the FHWA categorization of the condition of a bridge deck, (see II-C), the acceptable restoration procedures have been developed by FHWA for use on Federal-Aid projects as shown in Table 1 (2).

Bridge decks for reconstruction will be selected by the WSDOT Bridge Branch and districts and incorporated into the priority array (2). The selection of the method for reconstruction will be recommended by the Bridge and Structures Branch. Factors that will influence the decision include the following:

(a) the extent of chloride contamination
(b) the load-carrying capacity
(c) type of structure
(d) size and geometry of the structure
(e) impact of reconstruction on traffic flows
(f) cost and availability of funds
(g) remaining service life of structure

# Table 1.

## Acceptable Reconstruction Procedures for Federal-Aid Participation (2)

Testing Steps
1 **Visual**
2 **Delamination**
3 **Elect. Potential**
4 **Rebar Depth**
5 **Chloride Content**

| CATEGORY | PROCEDURES | ACCEPTABLE PERMANENT RECONSTRUCTION | EXPERIMENTAL COST EFFECTIVE RECONSTRUCTION (Estimated Extended Life 10 to 15 years) |
|---|---|---|---|
| Structurally Inadequate | | Complete Deck Replacement (Unless restorable) | |
| Extensive Deck Deterioration ① | Required Restoration Work | Complete Deck Replacement | Removal of all deteriorated concrete. Follow the repair procedure approved for the protective system selected. |
| | Testing | Steps #1 thru 5 as necessary (Probably only Steps #1 & 2) | Steps #1 & 2 only, except all steps on the first 3 plus 10% of the remaining decks |
| | Suggested Protective Systems | Epoxy Coated Rebars<br>Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection |
| Moderate Deck Deterioration ② | | Same as for Category #1 above<br>OR<br>Same as for Category #3 below as determined by the State | Same as for Category #1 above |
| Light Deck Deterioration ③ | Required Restoration Work | Removal and replacement of all areas of deterioration and chloride contaminated concrete as determined by corrosion potentials and/or chloride sampling. (Less than 5% of deck area) | Same as for Category #1 above.<br>Note: For this category of condition permanent restoration is strongly recommended. |
| | Testing | Steps #1 thru 5 | Steps #1 & 2 only, except all steps on the first 3 plus 10% of the remaining decks |
| | Suggested Protective Systems | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection |

## VI. DETERIORATION PREVENTION

In order to protect existing uncontaminated concrete bridge decks from reaching a chloride-ion content of 2 lb/cu yd (1.18 kg/cu m), which is considered to be the threshold of deterioration, WSDOT requires a protective system on the uncontaminated bridge decks. Two exceptions to this requirement are bridge decks with timber substructure and bridges approaching their life expectancy.

Among 654 bridges tested for chloride content in the State of Washington since 1980, approximately 150 bridges have shown a chloride content in excess of 2 lb/cu yd (1.18 kg/ cu m).

Potential candidates for protective systems will be selected by the Bridge Branch with preference shown toward Interstate routes, high salt usage and integral decks (2).

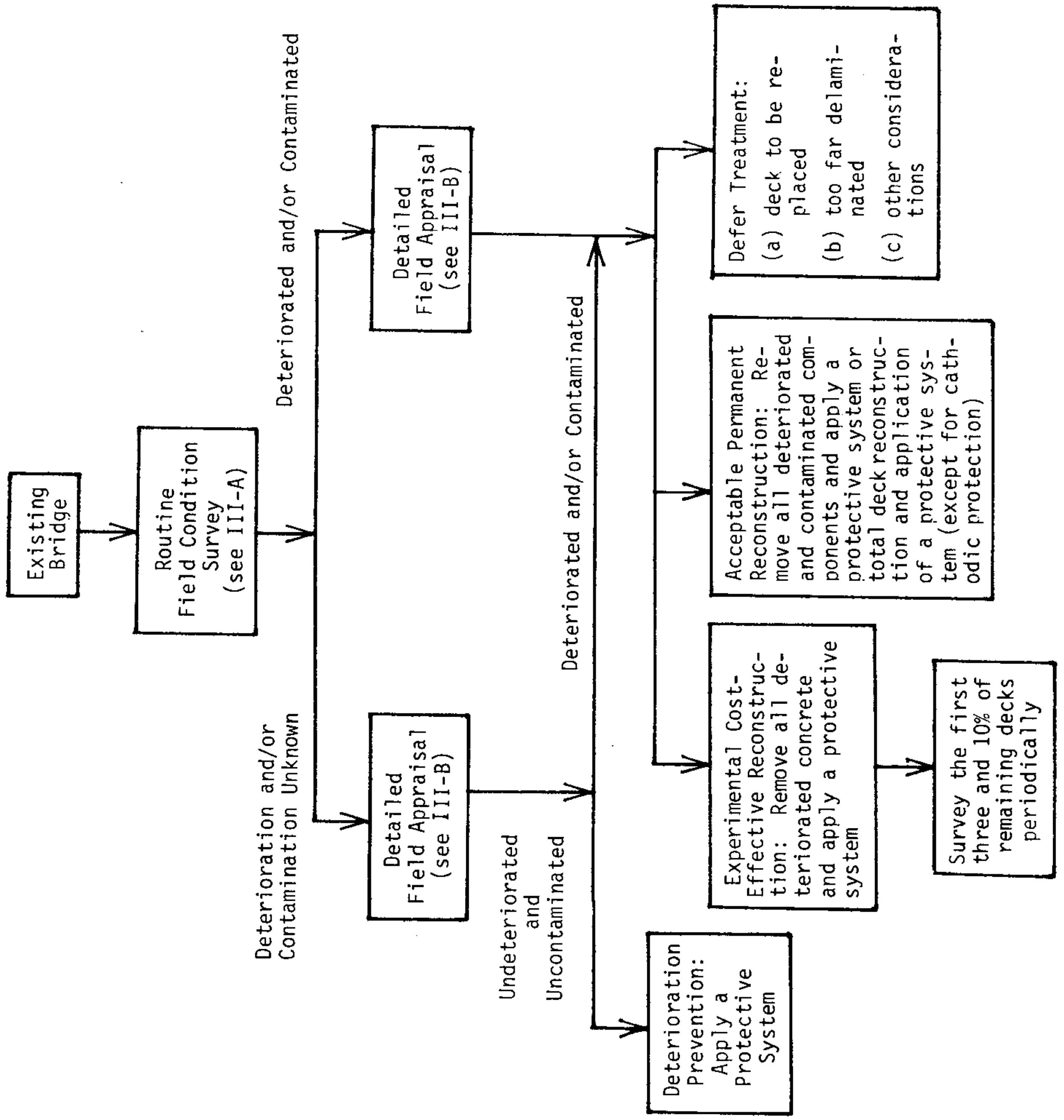
## VII. SUMMARY OF WSDOT'S DECK REHABILITATION AND PROTECTION PROCEDURE

The various activities involved in the WSDOT's concrete bridge deck rehabilitation and protection procedure can be summarized in the flow chart on the following page.

Discussion: As shown in the flowchart, the program generally offers two alternatives for rehabilitation of contaminated bridge decks (namely, permanent and experimental reconstruction) based on bridge condition surveys which are associated with four different protective systems (see Table 1). However, there are no criteria for reconstruction and installation of a protective system at different stages in the life of a bridge deck. Neither are there any criteria or justifications for selecting the optimum protective system from among four systems with different costs and service lives.

It is believed that guidelines are needed to determine whether it is more economical to reconstruct and place a protective system now at a given cost for a given expected additional life, or to perform minimal maintenance for a number of years and then replace the entire deck.

It appears that WSDOT (and FHWA as well) may need a systematic approach to evaluate and rehabilitate bridge decks. One possibility would be to organize the procedure into a "Bridge Deck Management System" (BDMS) patterned after the successful Pavement Management System (PMS) already developed in Washington by WSDOT. A suitably designed BDMS (or Bridge Management System, if the other bridge features were included) would enable the engineer to use collected data


Existing Bridge
Routine Field Condition Survey (see III-A)
Deteriorated and/or Contaminated
Deterioration and/or Contamination Unknown
Detailed Field Appraisal (see III-B)
Detailed Field Appraisal (see III-B)
Deteriorated and/or Contaminated
Undeteriorated and Uncontaminated
Defer Treatment:
(a) deck to be replaced
(b) too far delaminated
(c) other considerations
Acceptable Permanent Reconstruction: Remove all deteriorated and contaminated components and apply a protective system or total deck reconstruction and application of a protective system (except for cathodic protection)
Experimental Cost-Effective Reconstruction: Remove all deteriorated concrete and apply a protective system
Survey the first three and 10% of remaining decks periodically
Deterioration Prevention: Apply a Protective System

on the condition of bridge decks, along with available criteria, to not only evaluate the present condition of the bridge, but after some experience and history could begin to predict the rate of deterioration. This information would be a strong tool for management to make rational decisions about expenditure of available funds. A very important aspect of this system is the feedback on how well new as well as rehabilitated bridges are performing. Without this feedback, meaningful criteria would be difficult if not impossible to develop.

## VIII. CURRENT WSDOT CONCRETE BRIDGE DECK PROTECTIVE SYSTEMS

In view of the recognized need for protective systems on the bridges subject to deicing chemicals, WSDOT recommends the following alternative protective systems for new bridge deck construction, as well as rehabilitation of existing bridges. The protective systems, however, are not a substitute for quality design and construction. Of the deck protective systems described below, systems 1 and 4 will normally be specified for new bridges, while systems 2 and 3 will be used primarily for bridges requiring a leveling surface and for reconstruction of existing bridges. System 2 will be preferable to system 3, except where a bridge widening or reconstructing involves an existing asphalt overlay.

### A. System 1: Epoxy Coated Reinforcing Steel

This system includes 2.5 in (64 mm) of concrete cover over an epoxy coated top mat of reinforcing with no overlay (see Figure 1).

Discussion: Coating reinforcing steel with protective materials such as epoxy is a practical way to prevent the rapid corrosion of the bars, since it requires very little change in construction procedure. A stable coating can prevent corrosion by isolating the steel from water and chloride ions (1).

In their fabrication, reinforcing bars are bent to specific shapes, so cracking and disbonding of coating can take place on the area of the bars that is under tension during the bending. An alternative method might be to coat prefabricated reinforcing bars. Epoxy films on reinforcing bars can also withstand a moderate amount of abuse during their handling. Present handling methods should be modified and extensively damaged areas should be repaired with an approved material after being placed in the forms just prior to casting the concrete (18).

It has been reported (7) that a potential problem may exist when damaged epoxy-coated rebars in chloride-contaminated concrete (top mat) are electrically coupled to large quantities of uncoated steel in chloride-free concrete (bottom mat). It is shown (7) that in this case, a macro-corrosion cell with a large cathode (bottom mat) could develop and drive corrosion rapidly at damaged areas on the top rebar mat. FHWA studies have shown that the development of macro-corrosion cells appears to be of practical concern only when the total bare area exceeds 0.25%. Epoxy coating both top and bottom mats, on the other hand, could provide an added protection by electrically isolating the rebars. In this case, both the anode and the cathode sites are limited to the damaged area, and because of small cathode surface area, the rate of corrosion will be low. According to FHWA studies, up to 2% bare area would be appropriate for this case.

### B. System 2: Dense Concrete or Latex-Modified Concrete Overlays

One and one-half inches (38 mm) concrete cover over an uncoated top mat of reinforcing with 2 in (51 mm) of dense concrete overlay or 1.5 in (38 mm) of latex-modified concrete overlay may be used, at the contractor's option (see Figure 2). For new bridges, the overlay will be in place prior to opening the structure to traffic and prior to use of any deicing chemicals.

Discussion: Since 1980, eight bridges (seven existing, one new) have received dense concrete overlays and 20 bridges (14 existing, six new) have received latex-modified concrete overlays in the State of Washington.

According to FHWA, these protective systems have shown satisfactory service to date, but they cannot be considered as effective as epoxy coated rebars. Research has indicated that chloride penetration is retarded using these systems but not prevented. It has been reported (2) that this and other installation problems, such as cracking, delaminations and texturing difficulties, have prompted some states to invoke a moratorium on use of these systems for new decks. However, as indicated by FHWA, these protective systems, although not without problems, are considered practical and reasonably reliable for protecting existing decks, and field installations are performing satisfactorily after 10 to 15 years of service.

The relative advantages and disadvantages of concrete overlay (dense or latex-modified) are shown in the following table as reported in the NCHRP project, "Durability of Concrete Bridge Decks" (1):

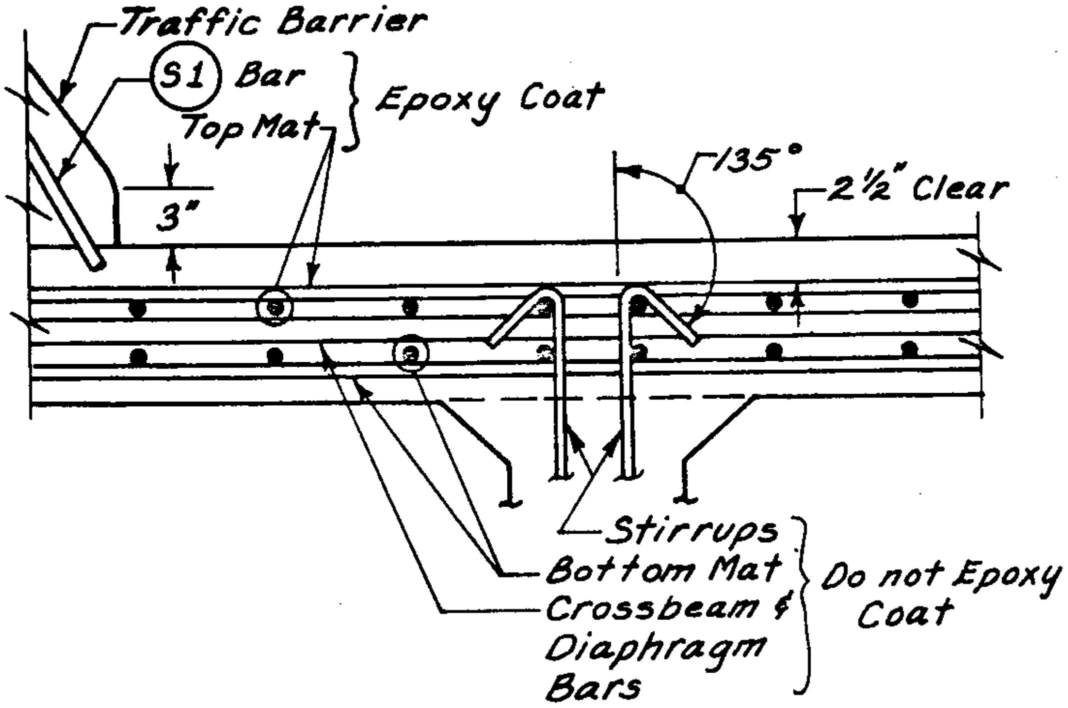


Figure 1.
WSDOT Procedure for Epoxy Coating Reinforcing Steel (2)

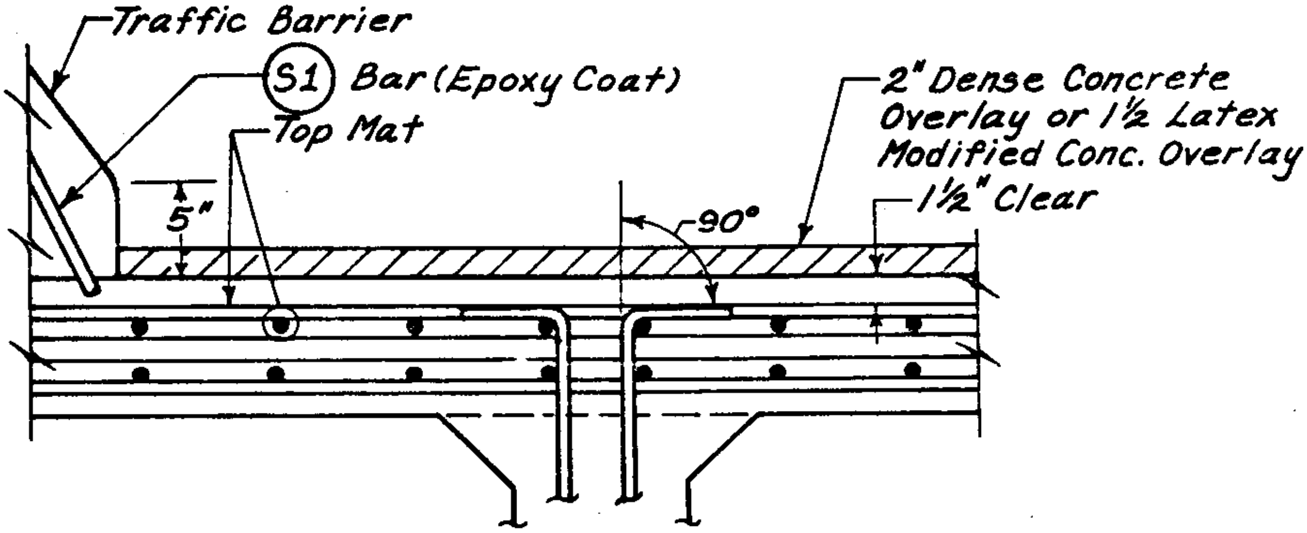


Figure 2.
WSDOT Procedure for Dense or Latex-Modified
Concrete Overlays (2)

Advantages

a. Structural component of deck slab
b. Relatively impermeable
c. Relatively long service life
d. Well-suited to repair of badly spalled or scaled decks
e. Many qualified contractors

Disadvantages

a. Not suited to decks with complex geometry
b. Cannot bridge moving cracks
c. Difficult to provide good skid resistance
d. May not stop active corrosion

### C. System 3: Asphalt Overlay with Waterproofing Membrane

A 2 in (51 mm) concrete cover over an uncoated top mat of reinforcing with a waterproofing membrane and asphalt overlay are recommended. Overlay thickness should be 0.15 ft (46 mm) (see Figure 3). The placement of overlay for new bridges will be prior to opening the structure to traffic and prior to the use of any deicing chemicals.

Discussion: Since 1980, 116 bridges have received this type of protective system in the State of Washington.

This system is recommended by FHWA only for existing bridges. As indicated by FHWA (2), with judicious selection of the membrane and careful attention to installation, the system can be a viable protective system. It has been reported (2) that the major disadvantage of this system is that the AC overlay experiences fairly rapid deterioration on bridges carrying high traffic volumes. Therefore, FHWA recommends using a more durable overlay system on high traffic volume (greater than 10,000 ADT) bridges.

NCHRP Project "Durability of Concrete Bridge Decks" (1) has reported the advantages and disadvantages of asphalt overlay and waterproofing membrane as follows:

Advantages

a. Bridges moving cracks
b. Relatively impermeable
c. Provides good riding surface
d. Applicable to any deck geometry
e. Many qualified contractors

Disadvantages

a. Performance highly variable
b. Will not stop active corrosion
c. Service life limited by wearing course
d. Nonstructural component of deck slab
e. Not suitable for grades in excess of 4 per cent

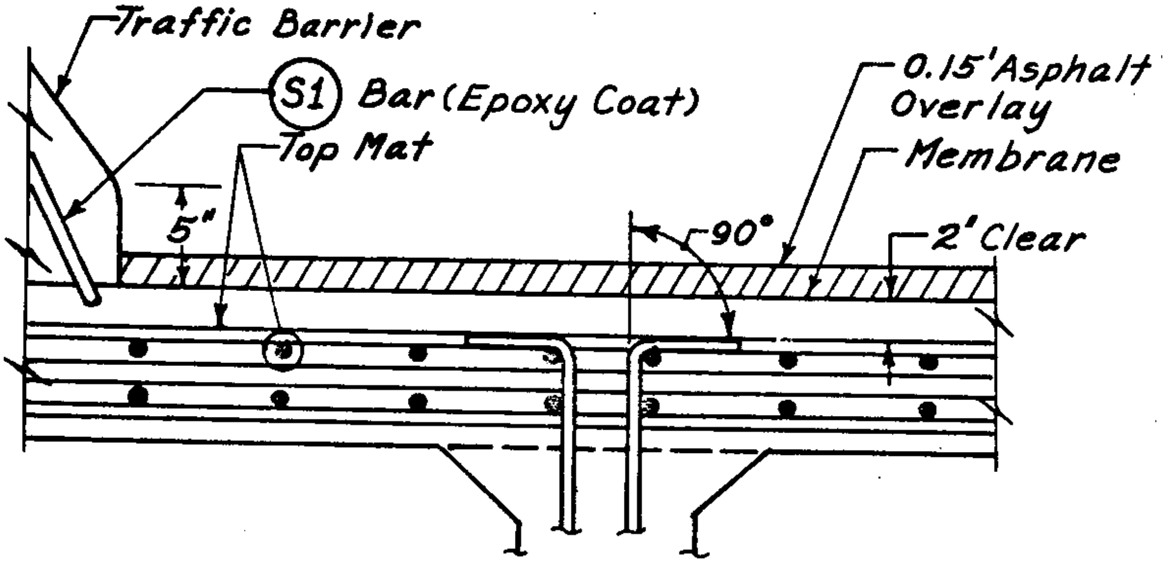


Figure 3.

WSDOT Procedure for Asphalt Overlay with Waterproofing Membrane (2)

### D. System 4: Two-system Corrosion Protection

For structures with transverse post-tensioning in the deck where deterioration would seriously impair the structure's integrity and deck restoration would be complex and costly, a combination of two protective systems (system 2 or 3 with epoxy coated bars) will be used. This system will also be considered for other types of structures, particularly post-tensioned box girders. Factors that will influence the decision are the type and size of structure, geographic location, impact of future deck reconstruction on traffic flow and anticipated use of deicing chemicals.

Discussion: According to FHWA, this "dual protective system" is warranted for bridges where deck deterioration could compromise structural safety and repairs would be costly and complex or bridges carrying high traffic volumes (greater than 10,000 ADT) where traffic control would unduly increase repair costs. When deck removal and replacement are impractical (such as post-tensioned segmental), FHWA recommends epoxy coating of all deck reinforcing as a necessity (2).

## IX. CURRENT WSDOT CONCRETE BRIDGE DECK EVALUATING METHODS

In addition to field condition surveys (see III-A), currently only delamination detection and chloride content analysis are included in WSDOT's detailed field appraisal of concrete bridge decks. WSDOT procedures regarding these two methods are briefly discussed below.

### A. Delamination Detection

Chain dragging is the primary method used by WSDOT at the present time for detecting delaminations. The device used by WSDOT, referred to as the "chain drag," is a simple homemade tool (36). One-half inch diameter steel pipe is fabricated into a "T" shape, with wheels at the tips of the flanges. Six 3/8 in (10 mm) steel chains are fastened to the flanges of the "T". Pushing the chain drag along the deck results in a readily discernible change in pitch when delaminated areas are encountered. The boundaries of a delaminated area are defined with just two or three passes of the chain.

Discussion: Chain dragging has been found to be an accurate, efficient, simple and economical method to detect delaminations which may result from causes other than rebar corrosion. The chain drag could indicate existence of delaminations in locations where the hammer or Delamtect (for more details see 2.2.1. in Appendix A) could not.

## B. Chloride Content Measurement

In WSDOT procedures for chloride content measurement, concrete sampling is done by a rotary hammer with a 1 in (25 mm) diameter carbide-tipped percussion bit which obtains powdered samples from bridge decks for laboratory wet chemical analysis. The samples are taken by drilling a hole to a depth of 0.5 in (13 mm) above the level of the rebar and removing the powdered material before drilling the additional 0.5 in (13 mm) (see Figure 4). The procedure does not use a pachometer to determine the rebar location and depth. Instead, rebar depth is determined by (a) examining the original contract plans for the bridge, (b) examining the deck for exposed rebars and measuring the cover, or (c) drilling down to a rebar and measuring the depth. The location of the samples in WSDOT procedure is randomly selected and is spaced out over the entire width and length of the bridge, including shoulder areas and delaminated areas. Using this procedure, one sample is taken per 750 $ft^2$ (70 $m^2$) of bridge deck area with a minimum of 10 samples on any one bridge. A chloride content vs depth profile is also taken on a minimum of two samples per bridge. This is accomplished by taking additional samples above and below the rebar level sample as illustrated in Figure 5 (2).

Discussion: The use of a rotary hammer for concrete sampling has the advantage of portability, light weight, speed, and economy. However, the samples are not prepared (pulverized) under controlled laboratory conditions. Therefore, care must be taken not to contaminate samples at the sampling depth by abrading concrete from the sides of the hole (1).

A suggested sampling procedure is usually to locate the position and depth of the reinforcement using a pachometer to avoid drilling through reinforcing bars, and if the chloride content at the level of the rebar is required, a hole is drilled to the depth of the rebar minus 0.25 in (6 mm) and cleaned out with a vacuum cleaner. The hole is drilled for a further 0.5 in (13 mm) and the sample is collected for analysis (1,8).

In determining chloride content of the samples, the quantity of coarse aggregate in a sample will affect the quantity of chloride since the major

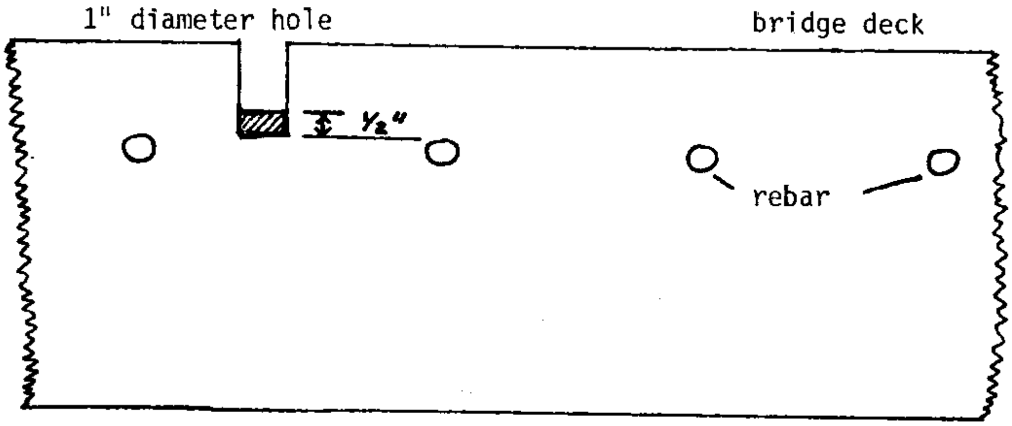


Figure 4.
WSDOT Sampling for Chloride Content at Rebar Level (2)

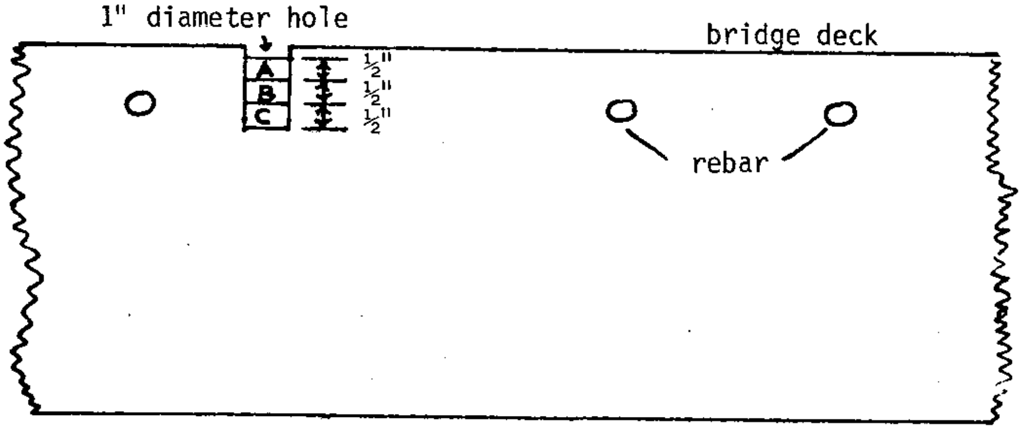


Figure 5.
WSDOT Sampling for Chloride Profile (2)

portion of the chloride is normally contained in the mortar phase. Thus, the sample must be representative or a correction must be applied to non-representative samples (for more details see 2.2.4.3 in Appendix A).

## X. WSDOT RESEARCH AND SPECIAL INVESTIGATION

WSDOT's research efforts are directed toward the following, as reported in the WSDOT bridge deck program (2):

(a) Reevaluation of the reliability of the half-cell tests.

(b) Performance of existing overlays over chloride contaminated decks.

(c) Incorporation of new materials and systems into original bridge designs.

(d) Continue to communicate with other states, research groups and the industry to keep abreast with the state-of-the-art.

### A. Half-Cell Corrosion Detection Test

Half-cell corrosion detection tests have been conducted by WSDOT as part of a research program to investigate bridge deck deterioration (36). For the purpose of the investigation a corrosion activity measuring instrument was constructed using a 3 ft (762 mm) long piece of 1.5 in (38 mm) diameter plastic pipe with a sponge tip at the lower end. The pipe was filled with a copper sulfate solution and a connecting copper rod inserted through a rubber top plug down to the sponge to complete the "saturated copper/copper sulfate half-cell." A second electrical connection was made to an exposed steel reinforcing bar and the potential measurements were made using a high input impedance electronic voltmeter. For interpretation of the half-cell readings, a dividing line between corrosion and non-corrosion was placed at -0.25 volts.

Readings were taken from 37 bridges and in general, the results indicated that local areas on many bridges show good correlation between high half-cell values, high chloride ion concentrations, and delaminated areas. As reported, delaminated areas nearly always coincided with areas of high half-cell potential, and that some correlation existed between areas of high activity and high chloride ion content. However, high chloride ion content did not necessarily indicate high half-cell potential and, similarly, high half-cell potential did not indicate that delamination had occurred.

The results of this study also showed that general conclusions regarding the entire deck could not, however, be reached by merely comparing values; and exact locations within a specific deck had to be known before attempts at correlation approached success.

Discussion: Half-cell electrical corrosion detection devices aid in delineating the areas of active corrosion at a given time (16). It is generally agreed (14) that a potential difference of -0.35 volts or greater is an indication that there is a greater than 90% probability that reinforcing steel corrosion is occurring in that area.

If it is assumed that chloride is the only cause of steel corrosion, then the areas of rebar corrosion should indicate high chloride content. However, a high chloride content will not necessarily be an indication of corrosion, since the corrosion may not have started yet. For delaminated areas, half-cell readings should be high, assuming that corrosion is the only cause of delamination. However, a high half-cell reading is not necessarily an indication of delamination, since delamination may not have started yet.

The above discussion leads to the point that if the chloride is the only cause of corrosion and if the corrosion is the only cause of delamination, then the half-cell test can be used as a partial substitute for chloride analysis and as a complete substitute for delamination detection to evaluate concrete bridge decks. However, if chloride is not the only cause of corrosion and/or if corrosion is not the only cause of delamination, then chloride analysis, half-cell readings, and delamination detection will not be able to individually fully define the extent of the problem, although they can be used in combination to validly define the total area of the problem.

## B. Internally Sealed Concrete

To evaluate effectiveness of wax beads as corrosion protection for steel in reinforced concrete bridge decks, WSDOT has tried a demonstration project on a twin bridge in King County. The objective of this study was to establish a test bridge deck which would contain a quantity of wax sufficient to fill all pores (or capillaries) in the top 2-in (51-mm) layer of concrete. One of the structures had wax introduced into the top 2-in (51 mm) layer of deck concrete during construction. The other structure contained no wax and was established as a "reference" structure to be used for comparison.

After the concrete of the wax-impregnated deck slab had cured and prior to opening to traffic, the deck was heated using an electrical blanket system so that the wax beads could melt and seal the pores in the concrete.

The internally sealed concrete overlay was opened to traffic in 1976. In 1978 some transverse cracks were inspected on the bridge. These cracks were within 30 ft (9.14 m) of each end of the bridge. In 1981 the results of chloride content tests on the wax bead bridge and the adjacent structure, used as the control, showed that the wax bead bridge had a higher chloride content at the rebar level than the control bridge.

Discussion: Laboratory tests (20) have indicated that internally sealed concrete is significantly less permeable than both latex-modified and low-slump dense concrete. Until the end of 1978, a total of 14 experimental internally sealed concrete overlays have been constructed in the United States. It has been reported (1) that the overlays appear to be performing well, and no problems, other than the initial cracking and the cracking that developed on some continuous structures after opening to traffic, have been identified. However, recently (1980) a crack survey was conducted on an internally sealed concrete deck in Texas (24) before and immediately after heating the deck, and no cracks were found that could be related to the heat treatment. There was one transverse crack across the deck over each interior bent, which was reported typical for deck slabs placed continuously over interior bents. Another survey on the same bridge was made approximately nine months after heating and before the bridge was opened to traffic. The results of this survey are shown in Figure 6. The cracks were reported (24) to be the result of shrinkage. It is believed that further investigation is needed to find the cause of this type of cracking of internally sealed concrete decks.

Apart from its performance, the economic feasibility of internally sealed concrete overlays remains doubtful using present heat treatment procedures. Research is continuing in an attempt to eliminate the heat treatment entirely by encapsulating sealant materials inside a wall material which would slowly degrade in the alkaline concrete environment, thus releasing the sealant after the concrete has cured (7).

### C. Corrosion Inhibitors

Calcium nitrite has been used as a corrosion inhibitor in a demonstration project at a bridge in Washington State. The material was used in the precast

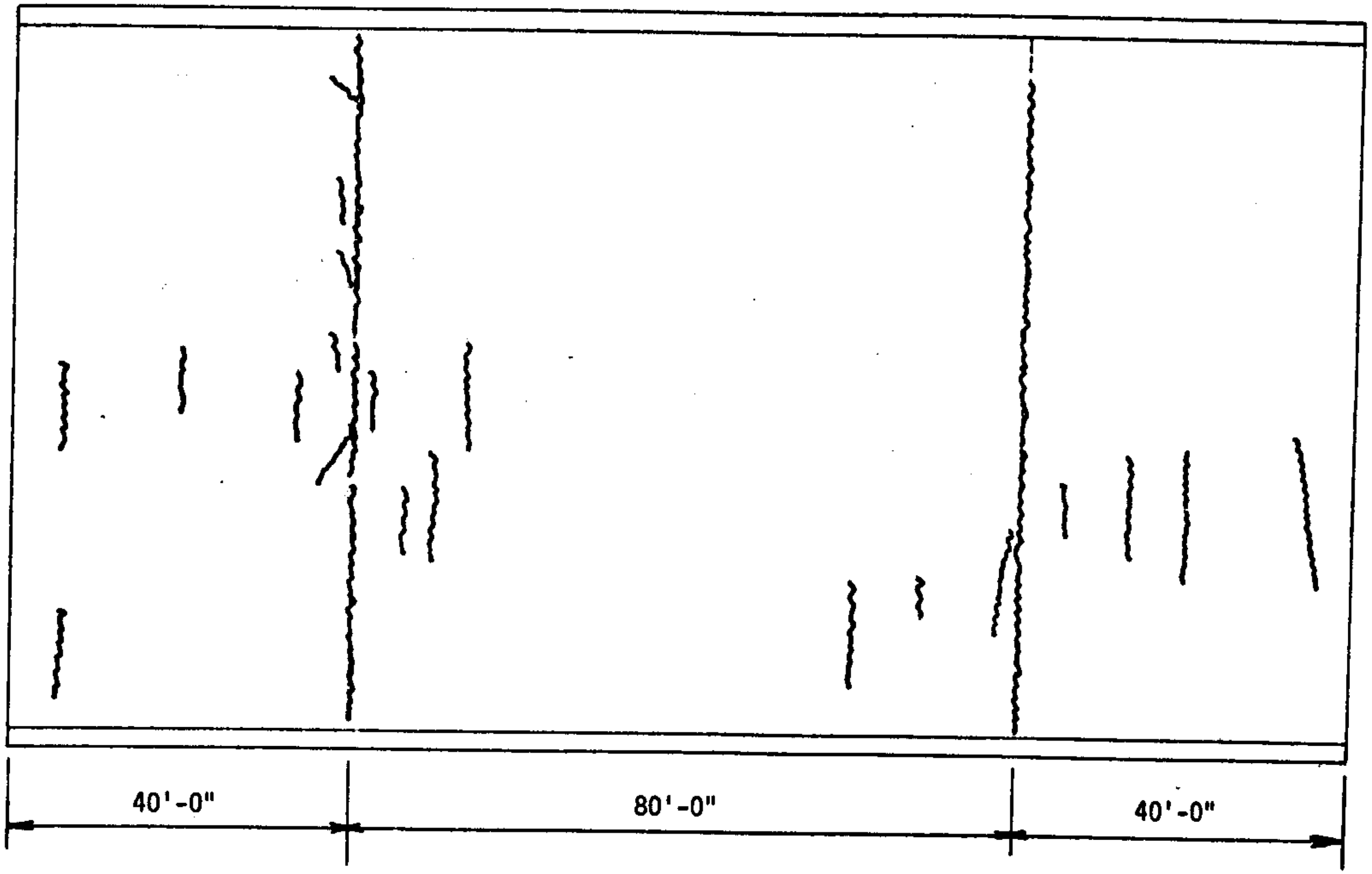


Figure 6.

Plan View of Deck Showing Results of Crack Survey
Made Nine Months After Heating
and Before Opening to Traffic (24)

prestressed piles and slabs by adding it to the concrete batch. This project is complete and a report will be forthcoming and a follow-up evaluation will be performed by WSDOT.

Discussion: Calcium nitrite seems to be a viable corrosion inhibitor for concrete. However, it has been reported (7) that there is a critical chloride to nitrite ratio above which corrosion could occur. Research is continuing to provide additional data about this ratio. The dosage rate of calcium nitrite, therefore, should be based on the critical ratio as well as the maximum chloride content expected at the rebar level during the design life of the structure.

## D. Cathodic Protection

WSDOT is considering a demonstration project using a cathodic protection system for acceptance by FHWA. This would include installation of an *in situ* impressed current cathodic protection system on a chloride-contaminated deck (2).

Discussion: Most techniques for rehabilitation discussed previously are basically delaying tactics; i.e., the damage caused by corrosion of the reinforcing steel will probably require replacement eventually, but the service life will be extended. The most likely method presently available to actually stop corrosion in existing bridge decks and prevent it in new decks is cathodic protection.

The basic hypothesis of cathodic protection is that all galvanic corrosion on a structure has been halted when all points on its surface have been polarized to a potential equal to, or more anodic than, the open-circuit potential of the most anodic point on the structure (5). In this condition, corroding anodes on the steel are prevented from discharging ions. Thus, the current-discharging anodes become current-receiving cathodes; hence, the term cathodic protection (1).

No bridge in Washington has been treated in this manner, but a discussion of the technology is included in the state-of-the-art report (Appendix A). The following is a brief description of advantages and disadvantages of cathodic protection as stated in reference 1:

Advantages

a. Stops active corrosion
b. Can be used on decks with moving cracks
c. Provides good riding surface
d. Applicable to any deck geometry

Disadvantages

a. Presence of wearing course may accelerate deterioration of the concrete
b. Nonstructural component of the deck slab
c. Continuing maintenance procedure
d. Limited performance history
e. Service life limited by wearing course
f. Specialized contractor and inspection required
g. Electrical power source required

## E. Rapid Chloride Permeability Test

WSDOT is investigating the effectiveness of rapid chloride permeability test in order to evaluate chloride permeability of new products and materials as well as existing concrete bridge decks.

Discussion: Previously, electrochemical removal of chlorides from concrete bridge decks had been the subject of research which had demonstrated it was possible to remove significant amounts of chloride ions from concrete by applying an electric field between the reinforcing steel (cathode) and a surface anode (34). Recently, this technique has been used as a chloride permeability test method by reversing the polarity, that is, by applying an electric field between the rebar (anode) and a surface cathode. In this case, chloride ions ($Cl^-$) having a negative charge would migrate into the concrete toward the rebar. Increasing chloride ion concentration will decrease the electrical resistivity of the concrete and as a result, the current will be increased. Additionally, at the end of the test, the concrete could be analyzed with wet chemicals to determine what amount of chloride had permeated the concrete. A measure of the increase in current in time could then be correlated with the amount of chloride entering the concrete (35). The test can be conducted in the laboratory in the form of 4-in (102-mm) diameter core specimens or in the field nondestructively in bridge decks (for more information, see 2.2.5 in Appendix A).

## F. Thermographic Survey for the Detection of Delaminations

A thermographic survey for delamination detection was conducted by WSDOT on the east portion of the Hood Canal Bridge in 1982. The technique is based on detecting infrared radiation (IR) emitted by the deck. The delaminations possess different heating/cooling rates than the main mass of concrete and therefore are detectable by their different IR emission rates under proper environmental radiation conditions.

The results of this investigation showed that the thermographic survey worked to some extent under ideal conditions for concrete delamination detection but was not competitive from a time or economic standpoint with presently used methods such as chain dragging.

Discussion: Infrared thermography is a promising method to rapidly detect delamination of bridge decks. Bringing present day equipment up to the full potential of the theory could make the delamination detection more economical than chain dragging in the future.

# CONCLUSIONS AND RECOMMENDATIONS

## I. PROTECTION AND REHABILITATION STRATEGY

### A. Deterioration Prevention

It is believed that existing uncontaminated concrete bridge decks which do not have a protective system and are subject to deterioration caused by salting should be protected from reaching the chloride content corrosion threshold by receiving a protective system. This could prevent future high cost rehabilitation and reconstruction. Priority should be given to bridges of high salt usage and bridges where deck deterioration could impair structural safety.

The WSDOT is actively engaged in a program to replace or minimize salting, and is participating in pooled fund research on the development of substitutes such as calcium magnesium acetate (CMA). It is suggested that this investigation remain a high priority part of the bridge deck program.

### B. Reconstruction

The decision to reconstruct the deteriorated deck immediately, or undertake continued maintenance and reconstruct later, or to completely replace the deck is a complex one and should be studied carefully. Each site should have its own solution and the most appropriate solution for any particular site can only be reached by a conditions survey of the structure to determine the extent of the deterioration and contamination, as well as an economical analysis of the alternative reconstruction procedures. Another factor which should be taken into consideration is the remaining service life of the structure. The deck reconstruction strategy should be compatible with the service life of the whole structure.

The above factors would be accounted for in a Bridge Deck Management System. The philosophy of WSDOT could be built into the system to reflect Washington conditions. A well-designed system, for example, could provide for alternatives based upon cost-effective methods. The alternative of experimental cost-effective reconstruction as defined in the WSDOT Bridge Deck Program that does not call for removal of all chloride-contaminated concrete may be acceptable, but only if it can be shown that permanent reconstruction is not economically justified. For example, the extent of contaminated concrete is usually not known in sufficient detail prior to reconstruction to enable lump sum bidding. Consequently, the actual cost of a project based on cost plus procedures that requires complete removal of contaminated concrete may far exceed the savings in terms of extended bridge life. Depending upon other factors, the economical choice may not be permanent reconstruction.

## II. DECK PROTECTIVE SYSTEMS FOR NEW BRIDGES

### A. Epoxy Coating Top Mat of Reinforcing Steel

This is a practical and economical solution which could also be considered effective based on previous experience. According to FHWA, most research and experience shows that decks protected with this system can be expected to have a long maintenance-free life. However, the system has the potential of developing an accelerated corrosion pattern of cracks, holes and damaged areas on the coating caused mainly by fabrication and handling of the coated bars. Repetitive loading could also cause a wearing away of the coating. Therefore, performance of this system should be watched closely by WSDOT. If the preponderance of data shows good performance, the system can also be considered an efficient solution to prevent deterioration of bridge decks. Epoxy coating both top and bottom mats, on the other hand, could provide an added protection.

### B. Dense Concrete and Latex-Modified Concrete Overlays

Although these protective methods have shown generally good performance, local bond failures and crackings have been reported. Like all rigid overlays, these systems are susceptible to cracking, especially on continuous structures. An efficient, maintenance-free, epoxy coating reinforcing steel, therefore, is believed to be preferable to these methods on new constructions.

### C. Asphalt Overlay with Waterproofing Membrane

Application of this system is not recommended on new bridges, since there might be problems with performance during its service life. Further investigation is suggested.

### D. Two-System Corrosion Protection

This is intended to mean epoxy-coated rebars in combination with dense concrete overlay, latex-modified concrete overlay, or membrane and AC overlay. This approach should be used only where exposure is very severe and the bridge is the most important transportation link or if the deck deterioration could compromise structural safety and the structure is very difficult to repair.

## III. DECK PROTECTIVE SYSTEMS

### A. Dense Concrete and Latex-Modified Concrete Overlays

Regardless of some local bond failures and crackings that have been reported for these protective methods, they have shown generally good performance and are believed to be practical for protecting existing bridge decks. These systems are well-suited for repair of badly spalled or scaled decks. However, they have a disadvantage in that when placed on existing cracks, they will quickly reflect those cracks.

### B. Internally Sealed Concrete Overlays

Although laboratory tests have indicated that the system is significantly less permeable than both dense concrete and latex-modified concrete overlays, its use is not recommended at the present time on new or existing bridges, since its economic feasibility remains doubtful using present heat treatment procedures. Development of cracking has also been reported on some bridges. Research is continuing in an attempt to eliminate the heat treatment entirely.

### C. Asphalt Overlay with Waterproofing Membrane

This is a relatively inexpensive choice compared to the dense and latex-modified concrete overlays. Because of this factor, it has been a frequent choice of WSDOT on existing bridges. It is convenient to overlay bridges at the same time other work such as the 4-R program is being done. As a result, well over 100 bridges in Washington have been sealed with a membrane and overlayed with asphalt, at an approximate average cost of about $15,000.

Although a large number of bridges have been treated, there appears to be very little data available to indicate the effectiveness of this system. For example, there may be problems with potential movement of the membrane, cracking caused by traffic loading or from weathering. These defects in the membrane may go undetected and cause concealed corrosion damage.

Because of the large investment to date and the continuing program to use this system, it is recommended that several representative bridges be studied to determine the effectiveness of the membrane plus AC overlay system.

### D. Cathodic Protection

This seems to be the only protective system, short of removal of chloride-contaminated concrete, that will stop active corrosion in bridge decks. Cathodic protection devices, costs and systems should be explored in detail and further evaluation of the system is recommended through installation in one pilot project. The FHWA demonstration project would appear to be an excellent opportunity to try it on at least one bridge.

## IV. FIELD EVALUATION TECHNIQUES

### A. Wet Chemical Chloride Analysis

Chloride analysis is a valid method to define areas of contaminated concrete; however, it is not recommended as the only procedure to be used to evaluate concrete bridge decks. Other evaluation techniques, such as reinforcing steel corrosion detection and delamination detection, are non-destructive and may be more rapid and economical than chloride analysis. These latter two techniques can define a portion of the area of problem on the deck, reducing the number of samples for more expensive chloride analysis. Aside from their economy, these procedures can indicate problem areas which might not be the result of chloride intrusion.

### B. Chain Dragging Delamination Detection

This is a simple, low-cost, and non-destructive method that merely detects delaminations which may result from causes other than rebar corrosion. Thus, its use in combination with chloride analysis and half-cell corrosion detection is recommended.

Research is in progress to enable the detection of delaminations by remote sensing through the application of such techniques as thermography. Use of these techniques in the future could be more economical than chain dragging and should be evaluated.

### C. Half-Cell Corrosion Detection

While half-cell corrosion detection seems to have most potential for adoption as a standard test to verify corrosion existence and its relative magnitude of extent, it has the disadvantage of operator error from poor connections, poor conductivity, etc. It is, in effect, a laboratory procedure that has been taken out into the field. It might provide confirmation that corrosion exists, but it does not indicate the potential for corrosion. Thus, it should be used in combination with chloride analysis to define the extent of the problem. Its advantages include speed and reduced damage to a structure. It is recommended that investigations be made to refine this technique and improve procedures for its use as a routine evaluation tool.

## D. Pachometer Survey

It is recommended that a pachometer survey be included in the evaluation of bare concrete decks to determine if observed deterioration is related to insufficient cover over the reinforcing steel. The pachometer survey is also needed to locate core samples for chloride analysis to avoid drilling through reinforcing bars.

## E. Rapid Chloride Permeability Test

The rapid chloride permeability test could be of great value in judging chloride permeability of new products, or existing concrete bridge decks. Concretes can be tested nondestructively either in the laboratory or in the field. The test could be used as a screening mechanism for proposed materials of different types. Nevertheless, it is of little value in evaluating bridge deck condition, degree of corrosion or amount of chloride at the level of the reinforcing bars.

## F. Bridge Deck Management System

A final recommendation is that WSDOT begin to develop a management system for bridge decks, with a goal of implementation as soon as possible.

# APPENDIX A

# STATE-OF-THE-ART REPORT

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# 1. NATURE OF THE PROBLEM

Three conditions of bridge deck deterioration are commonly identified: cracking, scaling and spalling. These conditions of deterioration are described in the NCHRP studies (1, 3, 4) as follows:

## 1.1. Cracking

Cracking is a characteristic of concrete caused by its low tensile strength and its relatively large volume changes due to changes in humidity and temperature. Cracks are not necessary for corrosion damage to occur in concrete. Corrosion of reinforcing steel can occur in uncracked, high quality concrete if there is little cover for reinforcing steel. It is generally believed that cracks will hasten corrosion of intercepted bars by facilitating the ingress of moisture, oxygen and chloride ions to the reinforcement (1).

## 1.2. Scaling

Scaling is the flaking of surface concrete, often accompanied by the loosening of surface aggregate. Scaling is mainly the result of frost deterioration of concrete. It also can be caused by chlorides without freezing. But scaling is especially severe when concrete is subject to freeze-thaw action in the presence of deicers. Scaling can be markedly reduced by air entrainment and by linseed oil treatment. Another factor accepted by most engineers is that scaling is decreased by a low water/cement ratio (3).

## 1.3. Spalling

The significance of cracking and scaling palls in comparison with the cost of repairing damage from corrosion spalling, which is the major deterioration of concrete bridge decks. Spalling is the separation and removal of the surface concrete caused by corrosion of the embedded reinforcing steel. Substantial thickness of concrete may be involved and, once initiated, it is difficult to halt the corrosion process and permanently repair the damage (1, 4). After the steel begins to corrode and before spalls are visible, horizontal cracks, or delaminations, occur at or above the level of the top reinforcing steel.

### 1.3.1. Corrosion of Steel in Concrete (1, 5, 6, 7)

The basic reason metal corrodes is that the corrosion products are chemically more stable than the metals themselves. For example, iron oxide is the basic ingredient of reinforcing steel and corrosion deposits (rust) on steel. It is generally believed that the corrosion of steel in concrete is the electrochemical reactions caused by salt-laden moisture in contact with the steel (5 6,). For such a reaction to occur, three basic elements are necessary: an anode, where corrosion takes place; a cathode, which does not corrode but maintains the ionic balance of the corrosion reactions; and an electrolyte, which is a solution capable of conducting electric current by ionic flow (1).

The corrosion process of reinforcing steel in concrete is stated in the NCHRP project, "Durability of Concrete Bridge Decks," (1) as follows:

Iron has a substantial tendency to enter into solution, thereby liberating electrons at the anode (equation 1):

$$\text{anodic reaction:}\quad Fe \rightarrow Fe^{++} + 2e^{-} \tag{1}$$

The remaining electrons move from the anodic area to the cathode and, provided oxygen and moisture are present, hydroxyl ions are formed (equation 2):

$$\text{cathodic reaction:}\quad O_2 + 2H_2O + 4e^{-} \rightarrow 4OH^{-} \tag{2}$$

Ferrous hydroxide is deposited at the anodes (equation 3), and this is usually converted to ferric hydroxide to produce the familiar reddish-brown rust (equation 4):

$$2Fe + 2H_2O + O_2 \rightarrow 2Fe(OH)_2 \tag{3}$$

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \tag{4}$$

The electrochemical process in reinforced concrete is caused by electrical potential differences that could occur at various places in the concrete at locations of differences in moisture content, oxygen concentration and cracking. In such concrete, a corrosion cell is established along a reinforcing bar. The distance between the anodic and cathodic areas on the bar may range from less than 1 in (25 mm) to more than 20 ft (6 m) (1).

Reinforcing steel in uncontaminated concrete normally shows resistance to corrosion because of the concrete's high pH value. The presence of soluble chlorides in concrete changes the pH characteristics of concrete and can destroy the passivity of the steel. Once a certain concentration of chloride ion is exceeded, corrosion may begin in the presence of oxygen and moisture (1).

In the FCP Annual Progress Report for 1980, "Cost Effective Rigid Construction and Rehabilitation in Adverse Environments" (7), it is shown that pH measurements at the anode sites were in the range of 4.8 to 6.0, which was acidic and much lower than that previously theorized for a conventional steel corrosion in concrete. The work has shown that in the presence of chlorides, high rates of corrosion could occur without the presence of oxygen at the anode. The anodic corrosion reaction in the presence of chloride has been theorized (7) as follows:

$$Fe^0 + 2Cl^- \rightarrow (Fe^{++} + 2Cl^-) + 2e^- \quad \text{followed by}$$

$$Fe^{++} (Cl_2) + (2H_2O) \rightarrow Fe(OH)_2 + 2H^+ + 2Cl^-$$

Accordingly, it is indicated that a major role of $Cl^-$ is to facilitate corrosion at the anode without the presence of molecular oxygen (through iron chloride complexing and hydrolysis) and the $H^+$ ion generated by the above reaction is the cause of the low anode pH's measured in corroding concrete. Another finding reported (7) is that the iron chloride reaction is self-perpetuating in that the $Cl^-$ originally responsible for the reaction is released for reuse when ferrous hydroxide is formed. It is also shown (7) that pH measurements at the cathode sites in reinforced concrete are typically in the range of 11.5 to 13.0, regardless of the chloride content of the concrete. Thus, it is theorized that the primary cathodic reaction in concrete is oxygen reduction $(O_2 + 2H_2) + 4e^- \rightarrow 4OH^-$ (7).

Finally, it is concluded (7) that based on the above explanations, even if the molecular oxygen is completely removed at the anode, rapid corrosion can continue through iron chloride complexing and hydrolysis, providing $O_2$ for reduction remains available at the cathode.

The following paragraph from the report (7) describes the hypothesis for rapid deterioration of bridge deck concrete due to reinforcing steel corrosion:

> "Microcells, of small magnitude and little consequence by themselves, are formed when a critical amount of chloride penetrates to the first level of the reinforcement within the concrete. This decreases the potential at the microcell sites to values more negative than -0.35 volts (CSE), thus creating a large potential difference between rebars in the $Cl^-$ bearing area and rebars in the Cl- free concrete (on both the top and bottom rebar mats). The magnitude of the potential difference is sufficient to overcome IR losses through several inches or more of wet concrete and thus permit oxygen reduction-induced current flow between large amounts of rebar in $Cl^-$ free concrete and the microcell sites. These currents rapidly reduce the pH to values of 5 to 6 (by iron chloride complexing and hydrolysis, which releases $H^+$ and $Cl^-$ ions) at locally corroding sites which happen to be located on paths of least resistance to the macro-cathode. This low anode pH is so different from that of nearby portions of the top-mat bar, which are also in $Cl^-$ bearing concrete (pH of 11.5 to 13), that a second powerful macro-cathode (which also feeds the original anode spots) is created. The result is rapid spotty corrosion

followed by concrete cracking at total iron losses as low as 0.5 to 2% of the rebar value. Further, only a small percentage (1/10 or less) of the total rebar surface area may be visibly corroded when initial cracking occurs."

### 1.3.2. Chloride Content Corrosion Threshold

After chloride ions penetrate into the concrete and migrate to the steel, they destroy the normal passivity of the steel in its alkaline environment when the chloride ion concentration reaches a certain threshold level. This may cause corrosion of the reinforcing steel and subsequent damage to concrete (9). Chloride is reported to be present in a normal hydrated portland cement concrete in essentially three forms (8):

(a) free chloride ion

(b) chloride bonded more or less strongly with calcium silicate hydrates

(c) chloride combined in definite compounds like the calcium aluminate chlorides, the analogous calcium ferrite chlorides, and calcium oxychloride

It is suggested (8) that combined chloride does not contribute to corrosion of the reinforcing steel in portland cement concrete, that free chloride can promote the corrosion process, and that bonded chloride may or may not be available to cause corrosion.

Generally, two types of chloride analysis can be used (8):

(a) measurement of the free or soluble chloride
(b) measurement of the total chloride

The first method (measurement of free or soluble chloride) could be the ideal procedure for determining chlorides that cause the corrosion. However, it is not easy to define the division between free and combined chloride, and the resulting value for free chloride is very dependent upon the extraction medium and time (8).

The second method determines the total chloride present in a specific portland cement concrete and defines the critical chloride content (threshold value ) based on the chloride which is readily available to the corrosion process (8). Because of the problems associated with the first method, the second approach has been selected by Berman (9) for the wet chemical analysis procedure.

As explained before, normal concrete is highly alkaline and offers very good protection for reinforcing steel. The presence of chlorides in concrete, on the other hand, reduces its alkalinity and thereby makes it less protective.

The chloride content corrosion threshold is therefore dependent on the cement content of concrete. Work in FHWA laboratories (8) showed that the total chloride content corrosion threshold would be 0.2% $Cl^-$ on a cement basis assuming 75% of chloride is soluble. Relating this 0.2% $Cl^-$ to a portland cement concrete with a cement factor of 7.0 (685 lbs cement per cu yd, or 388 kg per cu m), the total chloride content corrosion threshold is approximately 1.3 lbs of $Cl^-$ per cu yd of concrete (0.77 kg per cu m). For bridge deck concrete with a cement factor of 6.0 (564 lbs cement per cu yd, or 336 kg cement per cu m). The corresponding total chloride corrosion threshold would be 1.1 lbs $Cl^-$ per cu yd of concrete (6.65 kg/cu m).

The above thresholds refer to concrete surrounding the reinforcing steel. However, it is reported (8) that no data are available to show that the higher concentrations of chloride in the concrete above the reinforcing steel will not migrate downward with time. This point has led field engineers to be conservative when defining the chloride content corrosion threshold. It has been mentioned by field engineers (8) that less than 1.0 lb of chloride per cu yd of concrete (0.59 kg/cu m) will not cause steel corrosion, greater than 2.0 lbs per cu yd of concrete (1.8 kg/cu m) will cause serious steel corrosion, and 1.0 to 2.0 lbs per cu yd (0.59 to 1.18 kg/cu m) is a questionable area.

Chloride content corrosion threshold is usually defined as per cent by weight of concrete or expressed in terms of weight of chloride ion, sodium chloride or calcium chloride per cu yd (or cu m) of concrete. The conversion factors between these various units are given in Table A-1 (1).

### 1.3.3. Factors Affecting Chloride Permeability

There is a good relationship between water absorption and chloride intrusion characteristics of concrete; i.e., concrete is as permeable to chloride ions as it is to water (11). This means that if concrete is soaked in salt water, water and chloride ions will be absorbed at almost the same rate. Therefore, the permeability of concrete is a major factor affecting the corrosion of reinforcing steel. Also, concrete of low permeability is likely to have a low electrical conductivity, since less water can enter and remain (1).

Concrete permeability depends mainly upon water-cement ratio and degree of consolidation. The effects of water-cement ratio and degree of consolidation on the rate of ingress of chloride ions are shown in Figures A-1 and A-2, respectively (1). Concrete with a water-cement ratio of 0.40 resists penetration by deicing salts significantly better than concretes with water-cement

Table A-1. Conversion Factors for Expressing Chloride Content of Concrete (1)

| Required Units | Method of Conversion | Example(1) |
|---|---|---|
| 1. percent $Cl^-$ by weight of concrete | Result of test method | 0.025 |
| 2. ppm $Cl^-$ by weight of concrete | Multiply #1 by 10 000 | 250 |
| 3. percent $Cl^-$ by weight of cement | Multiply #1 by weight of concrete in $lb/yd^3$ ($kg/m^3$) and divide by cement content in $lb/yd^3$ ($kg/m^3$) | 0.15 |
| 4. lb $Cl^-$ per cu yd | Multiply #1 by weight of concrete in $lb/yd^3$ ($kg/m^3$) and divide by 100 | 1.0 $lb/yd^2$<br>or 0.59 $kg/m^3$ |
| 5. lb sodium chloride per cu yd of concrete ($kg/m^3$) | Multiply #4 by 1.65 | 1.65 $lb/yd^3$<br>or 0.97 $kg/m^3$ |
| 6. lb calcium chloride (2) per cu yd of concrete ($kg/m^3$) | Multiply #4 by 2.07 | 2.07 $lb/yd^3$<br>or 1.23 $kg/m^3$ |

(1) Assuming a cement content of 658 $lb/yd^3$ (390 $kg/m^3$) and concrete unit weight 145 $lb/ft^3$ (2320 $kg/m^3$).

(2) Calcium chloride dihydrate (flake calcium chloride), $CaCl_2 \cdot 2H_2O$

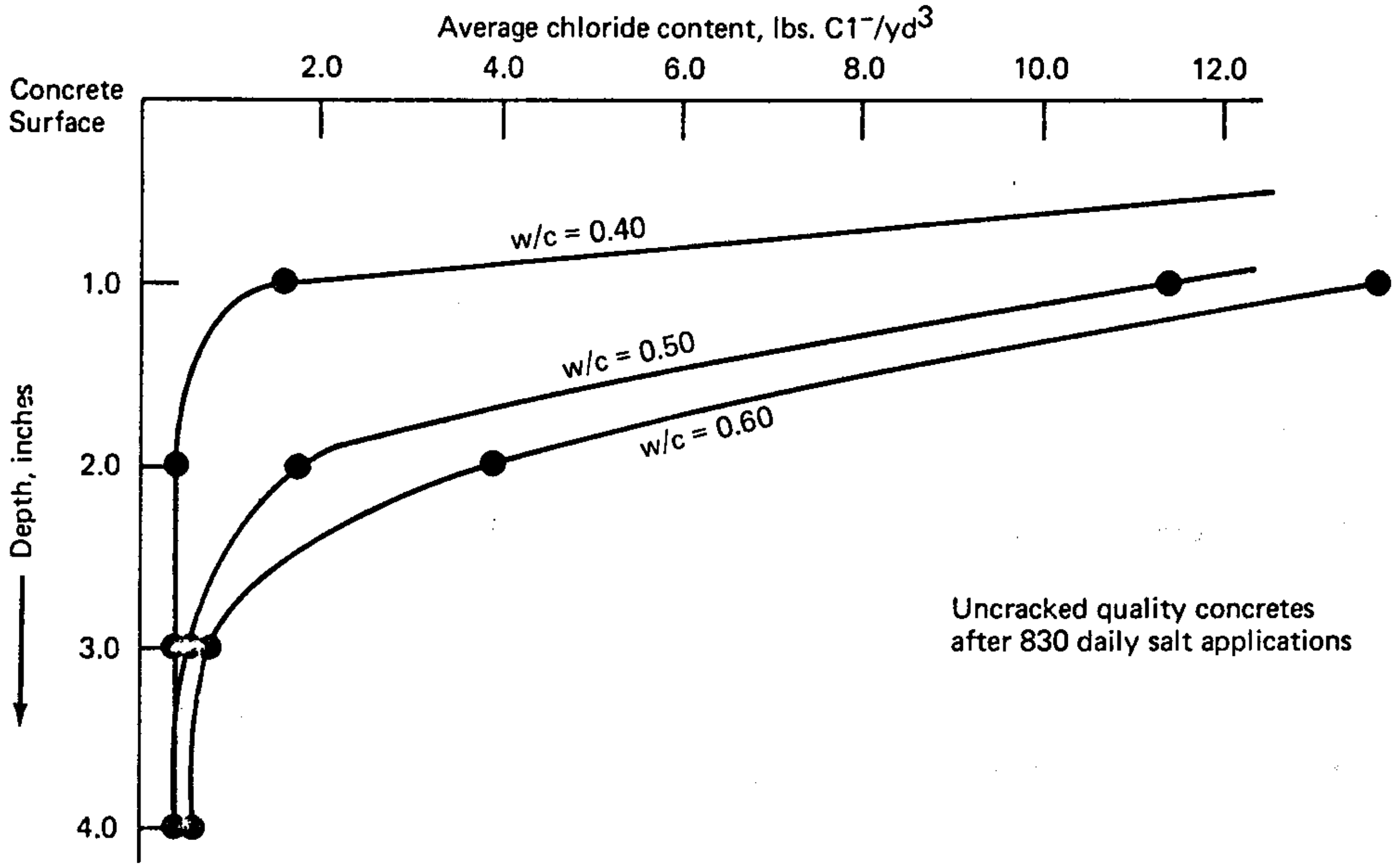

Figure A-1. Effect of Water-Cement Ratio - Average Chloride Profiles (10)

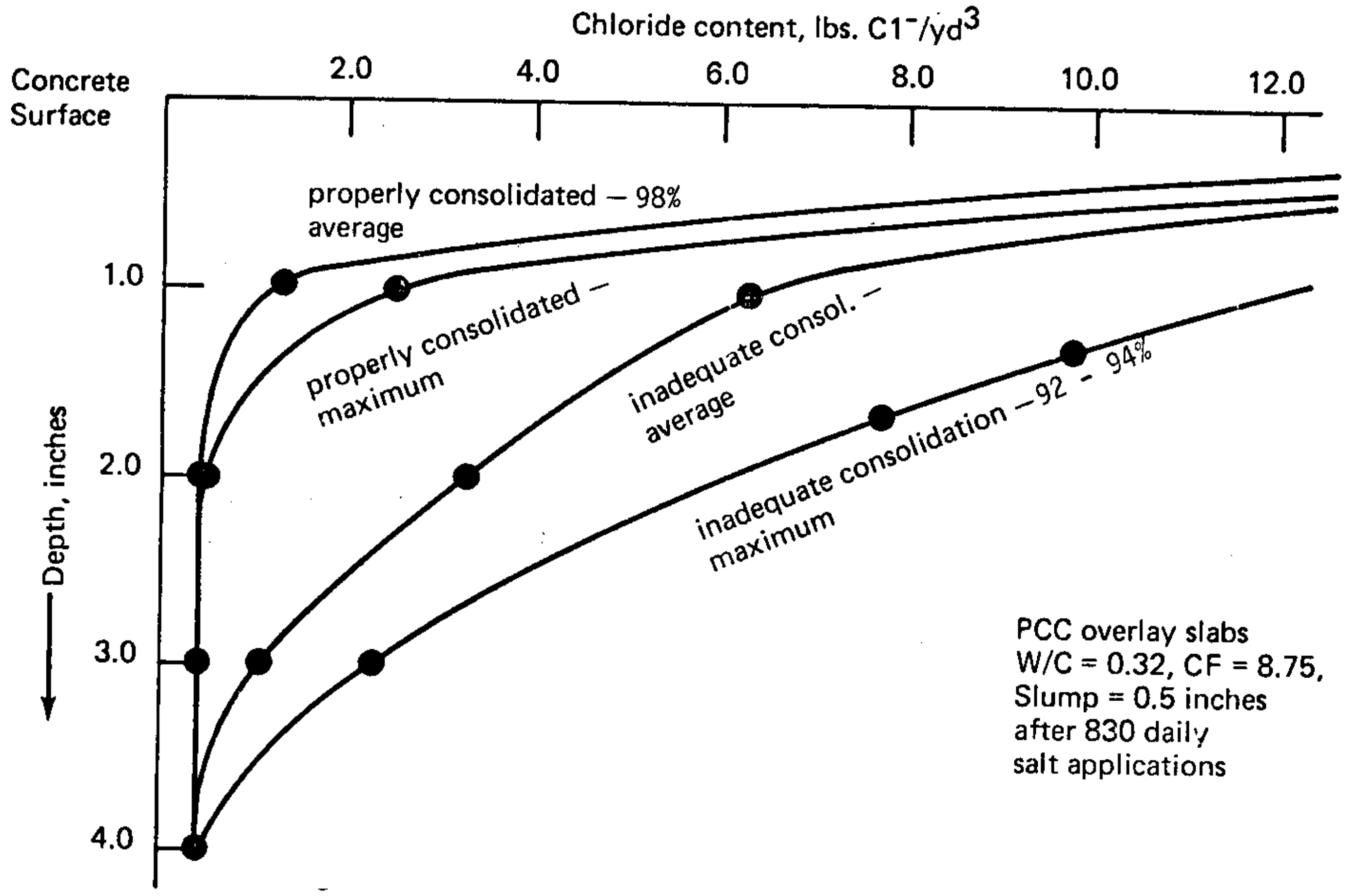

Figure A-2. Effect of Inadequate Consolidation - PCC Overlay Slabs (10) ratios of 0.50 and 0.60. A low water-cement ratio will not, however, ensure low permeability because, as shown in Figure A-2, without proper consolidation, concrete may be readily penetrated by chloride ions (the chloride profile for properly consolidated concrete in Figure A-2 is based on chloride analysis of samples for concrete with in-place density of about 98% of the concrete's rudded unit weight) (1, 10). An increase in cement content without decrease in water-cement ratio was found to have no apparent benefit for limiting chloride intrusion, as shown in Figure A-3 (10).

The increase in corrosion protection achieved with an increase in cover is more than a simple arithmetic relationship (1). The effect of depth of cover and water-cement ratio of the concrete, as determined in FHWA time-to-corrosion studies (10), is shown in Figure A-4. In the FHWA tests, the time to corrosion was determined by recording the number of daily applications of salt before active corrosion potentials were measured by the half-cell method. The American Concrete Institute (ACI) recommends a minimum of 2-in (50-mm) cover for bridge decks, if the water-cement ratio of the concrete is 0.40, and 2.5 in (65 mm) for a water-cement ratio of 0.45 (1).

### 1.3.4. Mechanics of Spalling (4)

NCHRP Project 20-5, "Concrete Bridge Deck Durability," investigated the mechanics of spalling. The following is a description of the spalling phenomenon:

> "Fresh concrete is cast in the deck, vibrated and screeded, and continues to consolidate. Aggergate particles settle to the bottom. Water is displaced and bleeds to the top surface. Meanwhile, the deck continues to deflect and vibrate as concrete is placed to complete the span, and the concrete subsides. Twin mats of reinforcement disrupt this subsidence, causing differential consolidation -- more settlement occurs between the bars than over them. The wetter the concrete, the more pronounced will be the difference.
>
> "Over a given bar, concrete is caused to separate as particles tend to flow to one side or the other to the lower settled areas between bars. Plastic shrinkage, drying shrinkage, and thermal stresses find relief in this area and cause a crack -- particularly if the steel is close to the surface and particularly if the concrete is very wet and thus subject to more shrinkage.
>
> "This crack over the topmost reinforcing bar, which is usually transverse to the traveled roadway, is very common in bridge decks. The concrete over the bar is, in accordance with accepted design procedures, considered to be cracked and is not used in design

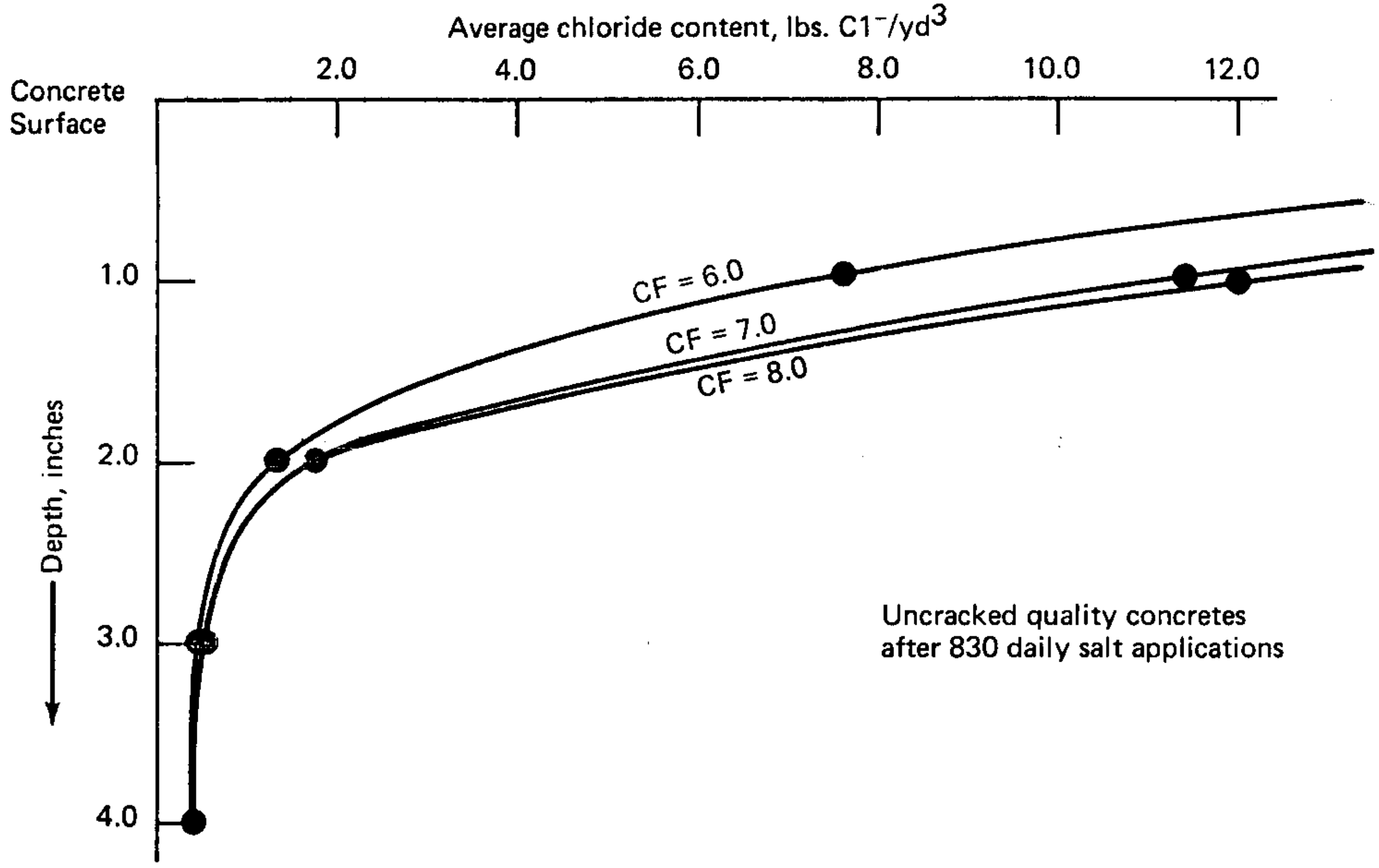

Figure A-3. Effect of Cement Content - Average Chloride Profiles (10)

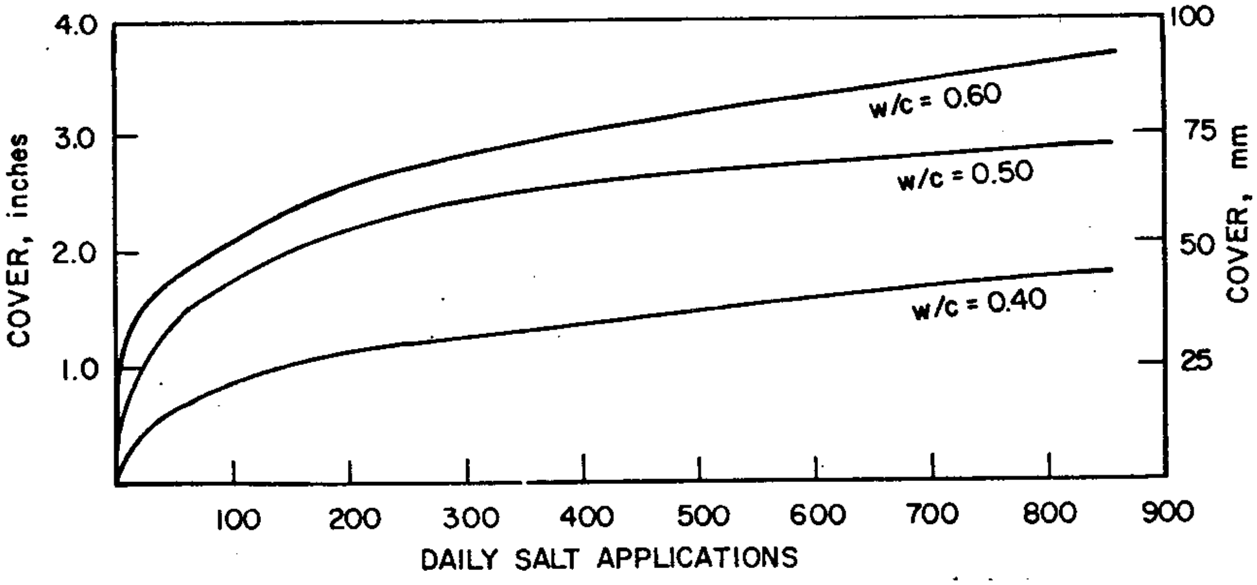

Figure A-4. Effect of Water-Cement Ratio and Depth of Cover on Relative Time to Corrosion (1) calculations. As years go by, water, dirt and salt wash into the cracks. This action is increased by the flexing action of the bridge superstructure (Figure A-5).

"Bleed water that is trapped under a surface crust creates planes of weakness that may cause early spalling.

"Even if there are no cracks, water can permeate porous concrete -- even air-entrained concrete -- and so can salt. This permeability is especially great in high-water-content concrete.

"Different concentrations of salt, or different concentrations of moisture, are sufficient to set up anodic and cathodic areas in a macrogalvanic electrochemical cell and actually cause a flow current. Salt in solution provides an electrolyte and oxygen in the water provides the oxidizing agent. An ideal environment is now established for the corrosion of the reinforcing steel, which involves the metamorphosis of iron into rust. The product of this corrosion occupies considerably more volume than the parent metal and results in a tensile force many times the strength of the concrete. When the cracks fill with water and freezing occurs, even greater pressures are exerted. Passing traffic creates significant compressive stresses, thereby inducing stress reversals and fatigue characteristics.

"Eventually certain random combinations of surface shrinkage cracks, cracked paste-aggregated bond, pressure from corrosion, and thermal stresses link to create a critical section that cracks to the surface. Ice pressure and the shear stress linked with bending movements soon extend this crack to form a complete fracture plane and a spall is created."

## 2. EVALUATING EXISTING BRIDGE DECKS

### 2.1. Field Condition Survey (2)

A field condition survey is needed to identify existing bridge decks that are possibly contaminated with deicing salts. The survey should also include an analysis of the extent of possible structural deficiencies. The information from the preliminary survey may be used to establish reconstruction priorities or to determine structural adequacy of the deck. The following are some examples of conditions recommended by FHWA (2) that can be readily determined or established, one or more of which may warrant reconstruction* and/or protective systems:

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*Reconstruction may include removal and replacement of all deteriorated and/or contaminated concrete, removal and replacement of all deteriorated reinforcing steel, or the complete removal and replacement of an entire bridge deck when necessary.

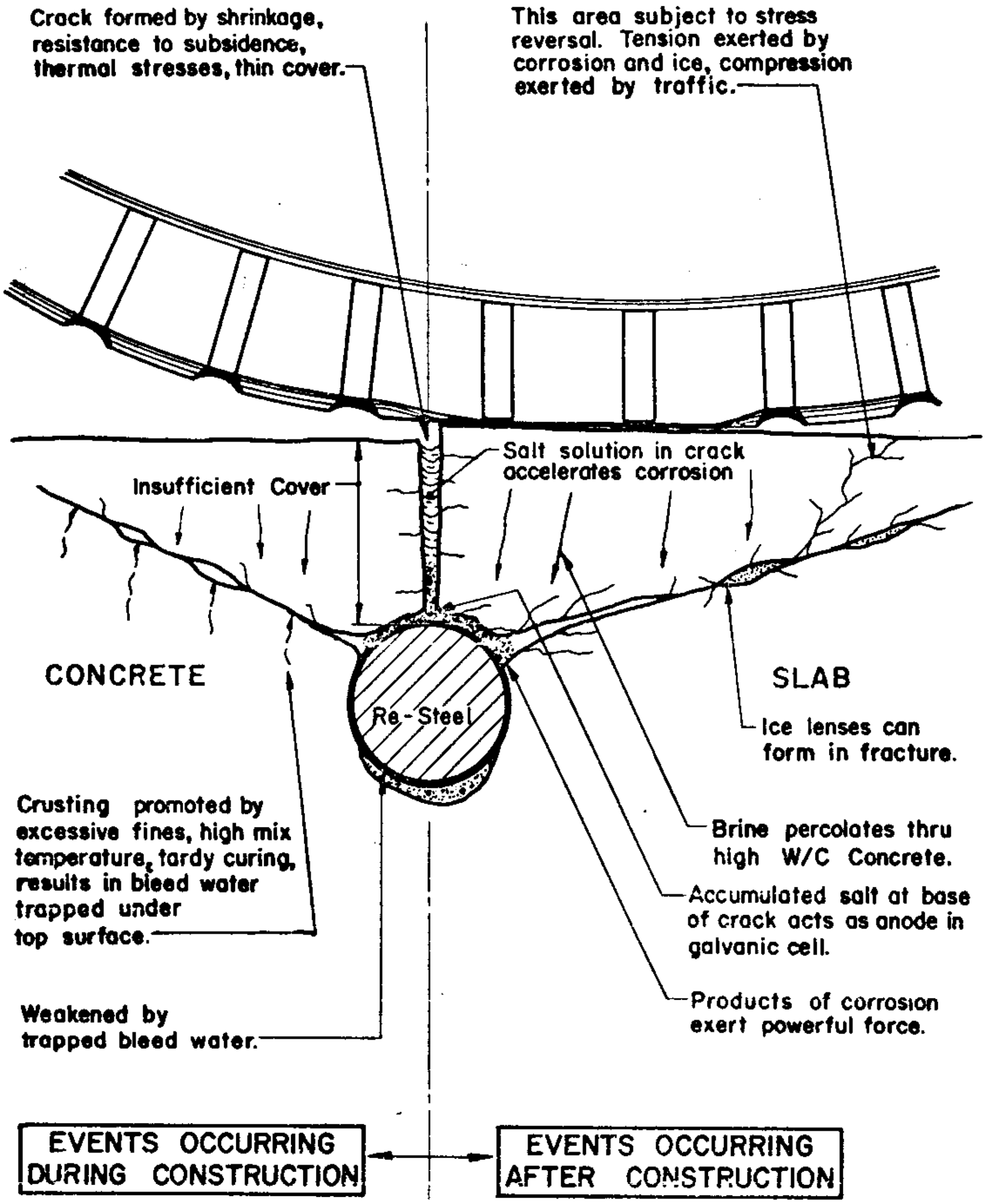


Figure A-5. Genesis of a Spall

(a) Maintenance records which indicate frequent application of deicing chemicals on this specific route.

(b) Visible concrete spalls which have occurred in the deck riding surface and/or evidence of unsound concrete in the bottom exposed surface of the deck slab which may indicate structural failure.

(c) Extensive deterioration of the asphaltic overlay logically due to underlying concrete deterioration.

(d) Evidence of delaminations (horizontal fracture planes) in the concrete deck.

(e) Evidence of reinforcing steel corrosion.

(f) Evidence of inadequate concrete cover over the reinforcing steel.

(g) Evidence of concrete bridge deck deterioration due to windblown salt water spray.

(h) When the structural adequacy of a bridge deck to carry current traffic loads is questioned, the field condition survey should also include a review and analysis of the extent of deficiencies and the feasibility of repair. Economics, traffic maintenance, etc., need to be evaluated when balancing the feasibility of structural restoration against complete replacement.

## 2.2. Detailed Field Appraisal

Where the field condition survey has shown that reconstruction and/or protective systems may be warranted, a detailed field appraisal should be made to further define the inadequacies of the existing deck. Based on the information from detailed field appraisals, an acceptable reconstruction procedure can be developed. The following, to the extent appropriate, are recommended by FHWA as the components of an evaluation system (2):

(a) Delamination detection with appropriate equipment to determine extent of internal fractures of the concrete.

(b) Determination of the extent of reinforcing steel corrosion by the use of a half-cell corrosion detection device.

(c) Determination of areas with inadequate concrete cover over the reinforcing steel by the use of appropriate equipment.

(d) Chemical analysis to determine extent of chloride contamination.

Using chloride analysis to determine the extent of chloride contamination as the only evaluation technique to define the concrete which must be removed for permanent reconstruction is not recommended. Other evaluation techniques, such as delamination detection to locate the extent of undersurface fractures, and reinforcing steel corrosion detection to define the areas of active steel corrosion, are non-destructive and could be more rapid and economical than chloride analysis. However, they are not able individually or in combination to fully define the areas of concrete removal when permanent reconstruction is planned. Still, they can validly define a portion of the concrete that must be removed, preventing the need for more expensive chloride analysis (8).

### 2.2.1. Delamination Detection

As explained earlier, after the steel begins to corrode and before spalls are visible on the deck surface, horizontal cracks, or delaminations, occur at or above the level of the top reinforcing steel. The separation between the upper layers of the deck concrete causes a dull sound to be heard when the deck surface is struck, which enables delaminated areas to be identified.

Tools devised for detecting delaminations are hammers, iron rods, and chains, and more recently, acoustical methods.

Chain dragging along the deck defines the delaminated areas. The boundaries of a delaminated area are defined with just two or three passes of the chain. The use of hammer and iron bar is tedious and time-consuming. The chain drag has been found to be accurate, efficient, simple and economical. The chain drag also could indicate the existence of delaminations in locations where the hammer method could not (1).

A commercially available portable device, known as the Delamtect, has been developed by the Texas Highway Department in cooperation with the Texas Transportation Institute for the detection of delaminated areas on a bridge deck (1). When the instrument is wheeled across the deck, it picks up and electronically interprets the acoustical signals generated by the instrument and reflected through the concrete. The equipment overcomes the workload of mapping out a full grid on the deck, which is used for chain dragging, and only the survey lines need be located. However, the detector does not locate delaminated areas as accurately as chain dragging; therefore, it is generally used when an overall indication of the deck condition is required (1).

Asphalt overlays thicker than about 1 in (25 mm) make the sound of delamination. Furthermore, on areas of discontinuity between the asphalt and concrete, it is not known if the discontinuity is a delamination or lack of bond between the asphalt and concrete (1).

Research is in progress to enable the detection of delaminated areas by remote sensing through the application of such techniques as thermography.

Studies by the Ontario Ministry of Transportation and the Virginia Highway and Transportation Research Council have indicated that infrared thermography is a promising method of detecting delaminations rapidly on bare concrete decks. It is reported (7) that under proper environmental conditions, delaminated areas are warmer or colder than the surrounding deck concrete. The optimum conditions as reported occur most frequently on hot sunny days with no wind.

### 2.2.2. Half-Cell Corrosion Detection

The method for measuring half-cell potentials on concrete bridge decks was developed in California (12). It was explained earlier that corrosion of steel in concrete is an electrochemical process. As the concrete becomes more contaminated with chlorides, the electrical potential of the reinforcing steel changes. Anodic and cathodic areas on the rebars are developed and the anodic areas begin to corrode. A potential difference between the anodic (half-cell) and cathodic (half-cell) may be measured by a voltmeter. However, this measurement would not be too meaningful, since the electrical activity of half-cells changes as the electrolyte changes. For this reason, a standard reference half-cell that retains a constant electrical potential is used to compare readings on a standard scale (13).

Earlier studies used a calomel cell as the reference, but the copper/copper sulfate cell is now preferred. The latter cell is reported to be sturdier and easier to use (1). It is comprised of a copper rod immersed within a tube in a saturated solution of copper sulfate. To compare the electrical potential of the copper rod to that of steel embedded in concrete, the two metals are connected through a voltmeter by making a positive electrical connection to the top mat of reinforcing steel. A moisture junction is also provided through the concrete to decrease the electrical resistance of the circuit between the copper rod and the reinforcing steel at the point at which the potential is being measured (see Figure A-6).

A full description of the test procedure has been published by ASTM (see Appendix B for ASTM C876). The procedure requires a direct electrical connection to the reinforcing steel except in cases where it can be documented that an exposed steel member is directly attached to the reinforcing steel (14).

Care should be taken to make low electrical resistance connections. If the measured value of the half-cell potential changes with time continuously, either the electrical resistance at the circuit is too great to obtain valid half-cell potential measurements or stray current from a nearby direct current system is affecting the readings (14).

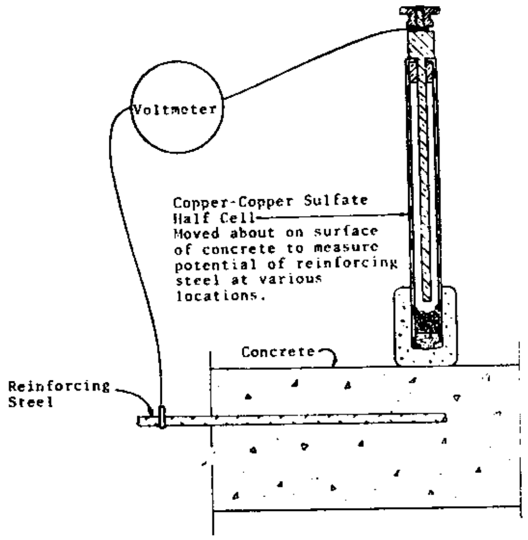


Figure A-6. Copper-Copper Sulfate Half-Cell Circuitry (14)

If a bridge deck has received a seal coat or a membrane, the seal coat or membrane must be punctured at the points of measurement to eliminate high electrical resistance of the circuit. This practice is also recommended when measurements are to be taken through an asphalt wearing course (1).

Half-cell potential values should be corrected for temperature if the half-cell temperature is outside the range of 72 ± 10°F (22.2 ± 5.5°C) (see Appendix B, ASTM C876, for temperature correction). The high electrical resistance of a frozen deck prevents completion of the circuit, and the test should therefore not be performed (1).

Although interpretation of the potential differences is disputed, it is generally agreed that a potential difference of -0.35 volt or greater is an indication of corrosive activity. The following are ASTM guidelines for interpretation of the potential differences. Voltages listed are referenced to the copper/copper sulfate (CSE) half-cell (14).

(a) If a potential reading is more negative than -0.35 volt, there is a greater than 90% probability that reinforcing steel corrosion is occurring in that area.

(b) Between -0.20 and -0.35 volts, there is a 50% chance that the steel is corroding.

(c) If a potential reading is less negative than -0.20 volt, there is a greater than 90% probability that no reinforcing steel corrosion is occurring in that area.

(d) Laboratory tests have shown that where potentials were more negative than -0.50 volt, approximately half of the concrete specimens crack due to corrosion activity.

(e) Positive readings, if obtained, are not valid and generally indicate insufficient moisture in the concrete.

### 2.2.3. Measurement of Concrete Cover

The chloride content of concrete bridge decks decreases rapidly with depth within the concrete (10). Thus, reinforcing steel with only a small amount of concrete cover will be much more susceptible to corrosion than steel placed deeper within the concrete.

A pachometer survey could be used in an evaluation of bare concrete decks to determine if observed deterioration is the result of insufficient cover to the reinforcing steel. A pachometer is an electronic device for nondestructively measuring the distance from the bridge surface to the top edge of the reinforcing steel. The apparatus employs the principle of magnetic flux (15) and has a battery, a hand-held probe, and a scale. The scale of the instrument is calibrated to record the distance between the probe and the bar directly. The scale of some instruments also includes a correction factor for the bar diameter (1).

In evaluating the coverage on bridge decks, use of the hand-held pachometer has been reported (15) to be a tedious and time-consuming process. Two men on their hands and knees record values from the scale as the probe is positioned. To expedite the process, the Oklahoma State Department of Highways envisioned a rolling pachometer that could be rolled across a bridge deck and would record the depth of cover on a strip chart recorder (15). The equipment essentially consists of a hand-held pachometer with additional electronic components.

The accuracy of most pachometers is reported to be in the range of ± 1/8 in (± 3 mm). In pachometer surveys, it should be kept in mind that measured values may be less than actual depths of cover, depending on existence of particles of magnetic iron in the concrete. A correction factor should therefore be established by either placing the probe on a sample of the concrete that does not contain any reinforcement or by coring and measuring the difference between recorded and actual values (1).

### 2.2.4. Measurement of Chloride Content

Measurement of chloride content is needed to determine the areas of a deck where the conditions for corrosion are present even though no rebar corrosion is occurring (16). The tools generally used to determine the condition of a concrete bridge deck with respect to rebar corrosion (i.e., delamination detectors and electrical corrosion detection devices) do not define the concrete in a potentially destructive condition. These tools may be sufficient if only a temporary reconstruction and protective system are desired.

#### 2.2.4.1. Location and number of samples for chloride analysis

The following procedure is recommended by Ken Clear (16) to determine the location and number of chloride analysis samples for documenting the condition of a bridge deck prior to permanent reconstruction. Primarily, visual surveys of the deck surface will define the spalled area. This measurement is not sufficient since each spall is only a product of the corrosion of reinforcing steel. Performing an electrical potential survey will then define the areas with potentials more negative than -0.35 volts (CSE) as the areas of active corrosion which should be designated as concrete removal sites. Subsequently, a delamination survey on the area of the deck with potentials less negative than -0.35 will determine the unbonded areas as concrete removal sites. Finally, the chloride content of rebar level concrete in the areas of the deck not yet designated for removal should be determined. The concrete with rebar level chlorides above the corrosion threshold is contaminated and must also be removed for permanent repair. The number of chloride determinations required will be determined based on the findings of visual, electrical potential, and delamination surveys (16). For more details, see Reference 8.

It is also recommended (16) that if electrical potential surveys cannot be validly performed on some bridge decks because the rebars are not electrically continuous or because of other factors, a visual survey, determination of the areas of delaminated concrete, and approximately 10 chloride determinations spaced across the deck in sound concrete could be used to provide initial understanding into the deck's condition. On this basis, areas for additional coring and chloride analysis can be defined (16).

#### 2.2.4.2. Sample acquisition (8, 16)

Conventionally, a truck-mounted, water-cooled coring rig has been used to obtain cores from a concrete bridge deck. The precise location of the core is determined through the use of a pachometer to avoid drilling through reinforcing bars. After the concrete cores are obtained they are sectioned to provide samples from the depths at which chloride content is desired. The concrete slices are then pulverized to provide the powdered samples required for complete extraction of the chloride during wet chemical analysis (8).

An alternative method is to provide a pulverized sample directly from the bridge deck by means of a rotary hammer. Coring (pulverizing) by this method is done without water so no soluble chlorides are removed from the concrete. An indicator attached to the hammer controls the coring depth, and a vacuum cleaner is used to remove unwanted pulverized material above the sampling depth. The hole is drilled further until the desired depth is reached. The powdered sample is then collected and is usually ready for analysis. The procedure can be used to obtain samples from any required depth (16).

The use of the rotary hammer has the advantages of portability, light weight, speed, and economy. The use of conventional core samples, on the other hand, permits the preparation of samples under controlled laboratory conditions and is generally preferred when maximum accuracy is required (1).

### 2.2.4.3. Methods of measuring chloride content

A method to measure chloride content of concrete was developed by Berman (17) and presented in a procedural form by Clear (8). The method employs a wet chemical analysis to determine the total chloride content of a concrete sample. The following steps show the test procedure as reported by Clear (8):

(a) Weighing a 3-gram sample of the powdered material.

(b) Adding distilled water and dilute nitric acid to dissolve the cement (and thus the chloride).

(c) Boiling.

(d) Filtering.

(e) Titrating with silver nitrate and recording successive additions of silver nitrate and the corresponding change in voltage. The voltage change is created by changes in the ionic potential between a chloride selective electrode and the solution containing the chloride extracted from the sample. The endpoint of the titration is the point of inflection of a curve of millivoltmeter readings vs volume of $AgNO_3$ solution added. The total per cent chloride is equal to:

$$\frac{35.453\ VN}{10\ W}$$

where: V is the volume of silver nitrate solution in ml, added up to the endpoint.

N is the normality of the silver nitrate solution.

W is the weight of the sample in grams.

Moisture in the concrete sample will have the effect of increasing the weight (W) and thus reducing the total per cent chloride reported. However, it has been shown that the effect of moisture in chloride content will be small (8) when compared with the repeatibility of the method (repeatibility is about 0.10 lb $Cl^-$/cu yd or 0.06 kg $Cl^-$/cu m) (16).

The quantity of coarse aggregate in a sample will also affect the quantity of chloride since the major portion of the chloride is normally contained in the mortar phase. If the $Cl^-$ content of a sample is used to estimate the chloride in a large mass of concrete, the sample must be representative or a correction must be applied to non-representative samples. It is reported (8) that the correction can be determined by measuring the weight loss of the sample between 221 and 932°F (105 and 500°C).

The aggregate in this range shows only a small weight loss, whereas the cement paste has a significant weight loss. Thus, the corrected chloride value can be expressed as (8):

$$\underset{\text{Ave}}{\text{Cl}^-} = \underset{\text{Sample}}{\text{Cl}^-} \left(\frac{\text{per cent loss ave.}}{\text{per cent loss sample}}\right)$$

The Kansas Department of Transportation had developed a method for in situ measurement of chloride content in bridge decks. The following test procedure is reported (1). A hole is drilled in the deck to a predetermined depth. The hole is filled with a borate nitrate solution, and a chloride-ion-specific electrode is inserted to measure the potential across the electrode. The potential is then converted to chloride concentration using a calibration curve. The accuracy of the method is about 0.5 lb $\text{Cl}^-$/cu yd (0.3 kg $\text{Cl}^-$/cu m)(1), whereas the repeatability of wet chemical analysis is about 0.1 lb $\text{Cl}^-$/cu yd (0.06 kg $\text{Cl}^-$/cu m)(16).

Recently an apparatus using two nuclear procedures, prompt capture gamma-ray analysis and neutron activation analysis, has been developed to measure the chloride content at the reinforcing steel level nondestructively. It has been reported (7) that the results of trials of the device on five Texas bridges have been very promising as compared to the wet chemistry method (7).

### 2.2.5 Rapid Chloride Permeability Test

Previously, electrochemical removal of chlorides from concrete bridge decks had been the subject of research which had demonstrated it was possible to remove significant amounts of chloride ions from concrete by applying an electric field between the reinforcing steel (cathode) and a surface anode (34). Recently, this technique has been used as a chloride permeability test method by reversing the polarity, that is, applying an electric field between the rebar (anode) and a surface cathode. In this case, chloride ions ($\text{Cl}^-$) having a negative charge would migrate into the concrete toward the rebar. Increasing chloride ion concentration will decrease the electrical resistivity of the concrete and as a result the current will be increased. Additionally, at the end of the test, the concrete could be analyzed with wet chemicals to determine the amount of chloride permeated the concrete. A measure of the increase in current with time could then be correlated with the amount of chloride entering the concrete (35).

A drawing of the apparatus used in the preliminary studies is shown in Figure A-7. A DC power supply applies a constant voltage of 80 volts between a copper screen (surface cathode) and steel reinforcing mat (anode) for a period of 6 hours. An example of the current flowing through the different types of concrete is shown in Figure A-8. As shown in the figure, up to 2.6 A of current are passed through a highly permeable concrete slab during the test period, compared with 1.8 A for a low permeability concrete slab.

By integrating the plots of current versus time the total amount of electric charge (in coulombs) passed through the concrete will be determined. Likewise, integrating the plots of chloride content versus depth of concrete will obtain a total concrete chloride content. Finally, concentration of the $NaCl_2$ solution remaining on the surface of the concrete after the test will give the chloride solution loss. Table A-2 shows some of the test results on different types of concrete with different permeabilities. As shown in the table, the magnitude of the current flow generally is proportional to the expected relative permeabilities of the various concretes (35).

Rapid chloride permeability tests can be conducted in the laboratory in the form of 4-in diameter core specimens or in the field nondestructively in bridge decks. The test is of a great value in judging chloride permeability of new materials and products. In addition to ordinary portland cement concrete, the procedure can be used with latex modified, internally sealed, low water-cement ratio, polymer impregnated, and polymer concretes (35).

## 2.3. FHWA Category Classification of Existing Bridge Decks

The limits describing three categories of condition as described below are recommended by the Federal Highway Administration (2) and can be used as guidelines. The limits should be established on past experience and cost factors. The categories described herein might overlap. In such cases the judgment should be based on engineering, economics and other factors to properly categorize an existing bridge deck (2).

### 2.3.1. Categroy 1 - Extensive Active Corrosion

5 per cent or over of the deck visibly spalled, OR:

40 per cent or over of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of nonduplicating areas consisting of the following: (a) spalls, (b) delaminations, (c) electrical potentials over 0.35 volts (CSE), and (d) chloride content samples greater than 2 pounds of chloride per cu yd (1.18 kg/cu m) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations and potentials more negative than 0.35 volts.

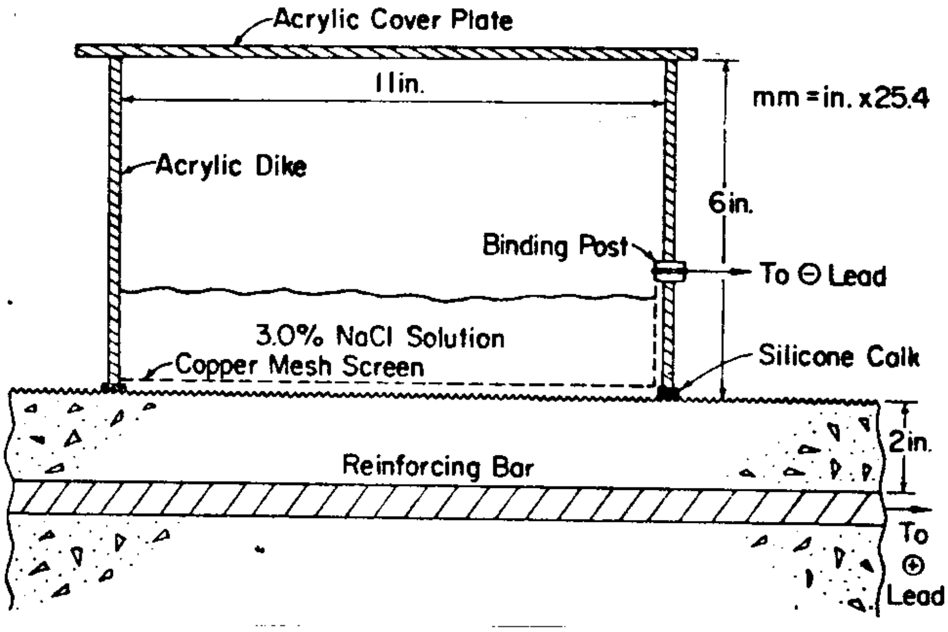


Figure A-7. Applied Voltage Apparatus Used on Slab Specimens (35)

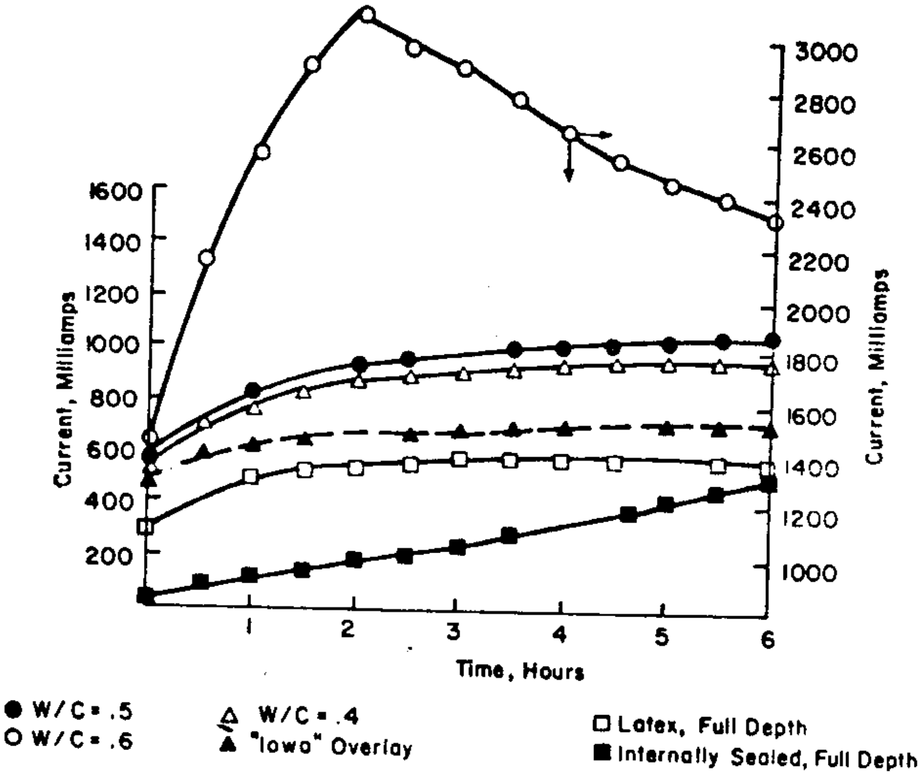


Figure A-8. Current versus Time for Various Slabs Tested at 80 Volts DC (35)

Table A-2. Permeability Characterization Parameters (35)

| Description | Total chloride in concrete[1] | Chloride solution loss[2] | Charge passed | Chloride permeability |
|---|---|---|---|---|
| | | *percent* | *coulombs* | |
| w/c = 0.60 | 0.77 | 0.53 | 52,570 | High |
| w/c = 0.50 | 0.47 | 0.38 | 22,500 | High |
| w/c = 0.40 | 0.37 | 0.34 | 20,410 | High |
| Latex overlay | 0.37 | 0.38 | 16,950 | Moderate |
| Latex full depth | 0.27 | 0.18 | 8,670 | Moderate |
| Iowa overlay | 0.31 | 0.27 | 15,270 | Moderate |
| Internally sealed full depth heated[3] | 0.10 | 0.10 | 5,770 | Low |
| Internally sealed full depth unheated | 0.93 | 0.53 | 36,070 | High |
| Internally sealed overlay heated[3] | 0.09 | 0.03 | 3,020 | Low |
| Internally sealed overlay unheated | 0.47 | 0.28 | 22,418 | High |
| Polymer impregnated | — | — | 0 | Very low |
| Polymer concrete overlay | — | — | 0 | Very low |

[1] Total chloride in concrete = Integral of chloride profile (concentration versus depth); that is, area under percent chloride by weight of concrete versus depth curves.

[2] Solution loss = Drop from initial 1.8 percent chloride (3 percent NaCl) concentration.

[3] Slabs stored in moist room for 90 days before testing; all others tested at initial "as-constructed" conditions.

### 2.3.2. Category 2 - Moderat Active Corrosion

0 to 5 per cent of the deck visibly spalled, OR:

5 to 40 per cent of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of nonduplicating areas consisting of the following: (a) spalls, (b) delaminations, (c) electrical potentials over 0.35 volts (CSE), and (d) chloride content samples greater than 2 pounds of chloride per cu yd (1.18 kg/cu m) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations and potentials more negative than 0.35 volts.

### 2.3.3. Category 3 - Light to No Active Corrosion

No visible spalls, OR:

0 to 5 per cent of the deck area having deteriorated and/or contaminated concrete or active rebar corrosion as indicated by a summation of nonduplicating areas consisting of the following: (a) delaminations, (b) electrical potentials over 0.35 volts (SE), and (c) chloride content samples greater than 2 pounds of chloride per cu yd (1.18 kg/cu m) of concrete as determined by 10 random samples of the deck area excluding the area of spalls, delaminations and potentials more negative than 0.35 volts.

### 2.3.4. Acceptable Restoration Procedures

Based on the foregoing categorization of the condition of the bridge deck, the acceptable restoration procedures have been developed by FHWA for use on Federal-aid projects as shown in Table A-3.

## 3. DECK PROTECTIVE SYSTEMS FOR NEW BRIDGES

### 3.1. Epoxy Coated Reinforcing Steel

Coating reinforcing steel with protective materials is a practical way to prevent the rapid corrosion of the bars since it requires very little change in construction procedure. A stable coating can prevent corrosion by isolating the steel from water and chloride ions (1). However, it is important that the coated bars be transported and fabricated without damage. The selection of suitable coatings has been the subject of several investigations (18). It is essential that the coating be durable in service environment and not impair the structural properties of the steel.

## Table A-3. Acceptable Reconstruction Procedures for Federal-Aid Participation (2)

| | | |
|---|---|---|
| Testing Steps | 1 | Visual |
| | 2 | Delamination |
| | 3 | Elect. Potential |
| | 4 | Rebar Depth |
| | 5 | Chloride Content |

| CATEGORY | PROCEDURES | ACCEPTABLE PERMANENT RECONSTRUCTION | EXPERIMENTAL COST EFFECTIVE RECONSTRUCTION (Estimated Extended Life 10 to 15 years) |
|---|---|---|---|
| Structurally Inadequate | | Complete Deck Replacement (Unless restorable) | |
| Extensive Deck Deterioration ① | Required Restoration Work | Complete Deck Replacement | Removal of all deteriorated concrete. Follow the repair procedure approved for the protective system selected. |
| | Testing | Steps #1 thru 5 as necessary (Probably only Steps #1 & 2) | Steps #1 & 2 only, except all steps on the first 3 plus 10% of the remaining decks |
| | Suggested Protective Systems | Epoxy Coated Rebars<br>Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection |
| Moderate Deck Deterioration ② | | Same as for Category #1 above<br>OR<br>Same as for Category #3 below as determined by the State | Same as for Category #1 above |
| Light Deck Deterioration ③ | Required Restoration Work | Removal and replacement of all areas of deterioration and chloride contaminated concrete as determined by corrosion potentials and/or chloride sampling. (Less than 5% of deck area) | Same as for Category #1 above.<br>Note: For this category of condition permanent restoration is strongly recommended. |
| | Testing | Steps #1 thru 5 | Steps #1 & 2 only, except all steps on the first 3 plus 10% of the remaining decks |
| | Suggested Protective Systems | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection | Two Course-Iowa System or Latex Modified Concrete<br>Membrane W/ac overlay<br>Cathodic Protection |

Coated reinforcing steel has been tested in the FHWA studies and it has been reported (7) that a potential problem may exist when damaged epoxy-coated rebars in chloride-contaminated concrete (top mat rebars) are electrically coupled to large quantities of black steel in chloride-free concrete (bottom mat rebars). It is shown (7) that in this case, a macro-corrosion cell (see Section 1.3.1) with a large cathode (black steel) could develop and drive corrosion rapidly at damaged areas on the top rebar mat; whereas epoxy coating both top and bottom rebar mats will lower the rate of corrosion since both anode and cathode sites will be limited only to the damaged area. FHWA studies have also indicated that for cases in which large quantities of black steel will be used in close proximity (within 2 feet) of epoxy-coated reinforcing steel, the development of macro-corrosion cells is a practical concern only when the total bare area exceeds 0.25%.

Nonmetallic-coated chairs and tie wires when used with epoxy coated rebars could be beneficial in reducing the frequency of top and bottom mat coupling. However, field tests indicated that these provisions eliminated coupling in only about 50% of the cases studied (7). It is also common practice to coat the bars in other elements, such as curbs and barrier or parapet walls, when protection against deicing salts is required. Use of coated bars does not, however, permit relaxation of the construction specifications for the remainder of the deck. The clear concrete cover over the bars should be at least 2 in (50 mm) to minimize the potential for corrosion at any bare area in the coating (1).

A detailed investigation of nonmetallic coatings was completed under the FHWA contract research program, "Nonmetallic Coatings for Concrete Reinforcing Bars" (18). The results of these studies may be briefly summarized as follows:

(a) The powder epoxies, in general, have better properties as barrier coatings than liquid epoxies; within the liquid epoxy series the solvent-free materials perform better than the solvent-containing systems. In general, the powder epoxy coatings yield films of more uniform thicknesses and with fewer holidays (pinholes normally not visually discernible) than the liquid epoxy coatings. However, the films of both powder and liquid epoxies have excessive numbers of holidays when their film thicknesses are in the range of 1-4 mils (25-102 μ).

(b) In their fabrication, reinforcing bars are bent to specific shapes. Cracking and disbonding of coating can take place on the area of the bars that is under tension during the bending. The epoxy coatings which do not perform well are either those with brittle epoxies or those with cured film thicknesses over 10 mils (254 μ). Another factor affecting the bending characteristics of coated reinforcing bars is the surface preparation of the substrate prior to application of the coatings. Disbondment can occur if epoxy coatings are applied to unprepared surfaces which are still covered with mill scale. Sandblasting rebars prior to coating prevents this type of coating failure. The temperature of the steel substrate during the epoxy coating process affects the flexibility of the cured epoxy. Higher flexibility of coatings can be achieved when rebars are heated and immediately coated. The present fabrication techniques for uncoated bars will certainly cause some damage to the epoxy coatings. The extent of such damage can probably be reduced by using bearing rollers, and bending wheels and anvils covered with pliable materials such as nylon. An alternate, and preferred, method might be to coat prefabricated reinforcing bars.

(c) Epoxy films on reinforcing bars can withstand a moderate amount of abuse. However, present handling methods should be modified, such as bundling coated bars together with nylon rope and protecting them from rough treatment at the construction site. Extensively damaged areas should be repaired with an approved material (such as a liquid epoxy) after being placed in the forms just prior to casting the concrete.

(d) An evaluation of pullout test results had indicated that epoxy-coated reinforcing bars have bond strengths essentially equal to uncoated bars when the film thicknesses are approximately 10 mils (254 $\mu$) or less. Both liquid and powder epoxies perform equally well, and the application does not significantly affect the bond strength of coated bars. In general, bond strength of coated bars is about 6% less than that of uncoated bars and is reported to be in the acceptable range.

(e) An evaluation of creep tests has shown that the epoxy coatings do not have a detrimental effect on the magnitude of the slip-time relationships developed with uncoated bars. However, the creep test seems to be more discriminating than the pullout tests.

(f) Considering flexibility, bond strength and creep characteristics, and minimum corrosion protective requirements, the optimum film thickness of epoxy films on steel reinforcing bars should be about 7 ±2 mils (178 + 51 $\mu$).

(g) Electrical potential measurements have not been found to be reliable indicators of the corrosion state of coated bars. A suggested method to monitor the corrosion condition of the reinforcing steel is electrical resistance measurements (see Appendix B for electrical resistance testing). Resistance measurements are probably more reliable indicators than potential measurements, since the resistant values are primarily dependent on the integrity of the coating films. The resistance of a film will sharply decrease if holidays develop or decrease more slowly if the film is gradually deteriorating overall. The resistance of the protective layer of water-soaked concrete is much lower than the resistance of a good protective coating on a bar.

### 3.1.1. Cost

As the use of epoxy-coated bars has substantially increased, the cost of the coated bars has decreased. The additional in-place cost of the coated bars was reported in most contracts as \$0.39/lb (\$0.86/kg), and \$0.15/lb (\$0.33/kg), for 1974 and 1977, respectively (1). Prices quoted are typically for #6 (19-mm diameter) bars. In the State of Washington, the cost of epoxy coating rebars has been reported as \$1.50/ft$^2$ (\$16.15/m$^2$) for 1982.

## 3.2. Corrosion Inhibitors

A corrosion inhibitor is generally an admixture to the concrete used to prevent the corrosion of reinforced steel. The mechanism of inhibition is complex, and there is no general theory applicable to all situations. Some inhibitors that appear to be chemically effective in preventing the corrosion of reinforcing steel cause a significant reduction in compressive strength of the concrete. Admixtures used to prevent corrosion of the steel by "waterproofing" the concrete, notably silicones, have been reported ineffective (1).

More recently, calcium nitrite has been reported (7, 19) to be a viable corrosion inhibitor. It was first used in Japan to facilitate the use of salt-bearing beach sands in reinforced concrete. To evaluate the corrosion inhibiting characteristics of calcium nitrite, various tests have been conducted. Generally, the information evolving from the tests points to calcium nitrite, in fact, as a viable corrosion inhibitor for concrete. However, it has been reported (7) that there is a critical chloride to nitrite ratio above which corrosion will occur. Research is continuing to provide additional data about this ratio. The dosage rate of calcium nitrite, therefore, should be based on the critical ratio as well as the maximum chloride content expected at the rebar level during the design life at the structure (7).

# 4. DECK PROTECTIVE SYSTEMS FOR NEW AND EXISTING BRIDGES

## 4.1. Concrete Overlays

Concrete overlays can be applied as the second stage of construction on a new deck, or in the rehabilitation of existing, deteriorated decks. In two-stage construction, the first lift of concrete is placed to cover the top of the reinforcing steel and the overlay usually is placed a few days later.

Potential advantages of concrete overlays when applied on a bridge deck are reported (1) as: (a) Properly-proportioned and consolidated concrete overlay can effectively retard the penetration of chloride ions. (b) Concrete overlays are well-suited to badly deteriorated decks because areas of concrete removal are placed during the paving operation, and because the overlay is a structural component, the load-carrying capacity of the deck slab is increased. (c) The concrete overlays are thermally compatible with the base concrete.

Concrete overlay has the inherent disadvantage that when placed on existing cracks, it will quickly reflect those cracks or stated differently, the existing crack is a stress raiser that causes that crack to continue to propagate which may limit concrete overlay's service life. However, in new construction, the cover is free from cracks directly over the reinforcing bars.

Epoxy, latex, and cement paste and mortar have all been investigated as bonding agents for construction of concrete overlays, and are reported (1) to perform satisfactory when the base is properly prepared.

Epoxy bond agents should be used on dry concrete surface whereas, latex bonding agents should be applied to a prewetted surface because the water helps penetration of the latex particles into the deck concrete and prevents the latex from drying. When using cement paste and mortar bonding agents ideally the deck should be kept wet for several hours before placing the overlay and the contact surface allowed to dry prior to concrete placement. However, on existing decks, where steel is exposed and the deck surface is irregular due to the preparation of base, a dry deck is preferable because the water causes rapid oxidation of sandblasted bars and it is also difficult to remove puddles of water from depressions in the deck surface (1).

Concrete overlays that have been used successfully on bridge decks include low slump, dense concrete, polymer modified concrete, and internally-sealed concrete.

### 4.1.1. Low Slump Concrete Overlays

The construction procedure includes the application of a very low water-cement ratio, dense, portland cement concrete overlay to provide impermeability and minimize shrinkage (see Section 1.3.3). The concrete must be properly air-entrained and proportioned to have a very low water-cement ratio, usually around 0.32. This is achieved by a cement factor of approximately 800 lb/cu yd (480 kg/cu m) and sufficient water to produce maximum slump of 1 in. (25 mm) (1).

The essential steps in construction of low slump concrete overlays when used in permanent bridge reconstruction are as follows:

(a) Scarify deck surface to remove all contaminants, such as oil drippings and linseed oil, from the surface concrete.

(b) Remove areas of deteriorated and contaminated concrete.

(c) Clean concrete surface by sandblasting.

(d) Apply a mortar bonding agent to the deck surface.

(e) Place the low-slump concrete overlay.

(f) Cure the concrete using burlap and water.

In two-stage construction, steps a, b and c are eliminated.

Low-slump concrete overlays use inexpensive materials but the placing operation is labor-intensive and requires proper consolidation of concrete and the use of specialized finishing machines designed for use with low slump concrete mixtures. As discussed in Section 1.3.3., a lower water-cement ratio without proper consolidation will not, however, ensure low permeability and without proper consolidation, concrete may be readily penetrated by chloride ions. The degree of consolidation should be checked with a nuclear density gauge; the requirement is a minimum consolidation of 98% of the concrete's rudded unit weight. Low-slump concrete finishing machines are much heavier than conventional finishing machines and are usually equipped with two screeds, at least one of which has considerable vibratory capacity. Good quality control and inspection procedures are required for the success of low-slump concrete overlays (1).

#### 4.1.1.1. Cost

Cost of concrete overlays in the past have been approximately \$4.00/ft$^2$ (\$43.06/ m$^2$), which does not include costs for repair work. Recent bids for concrete overlays indicate a downward trend in cost for concrete overlays. Costs as low as \$2.25/ft$^2$ (\$24.22/ m$^2$ ) have been experienced. It has been reported in one research effort (27) that when bridge decks with 3 in (76 mm) of cover and a water-cement ratio of 0.35 are compared with decks with 2 in (51 mm) of cover and a water-cement ratio of 0.44, life can be tripled for a 2% or less increase in construction cost.

### 4.1.2. Polymer-Modified Concrete Overlays

Polymer-modified concrete is made by incorporating polymer emulsions (approximately 15% solids by weight of cement) into the fresh concrete. The water of suspension in the emulsion hydrates the cement, and the polymer enters the structure of the concrete and provides supplementary binding due to its adhesive and cohesive properties. Polymer emulsions have been in use for a number of years, and the concrete is more commonly known as latex-modified concrete (1).

The construction procedures for latex-modified concrete, whether on new or existing decks, are very similar to those for low-slump concrete. The major differences in construction procedures are as follows:

(a) The bond coat usually consists of the mortar fraction of the latex-modified concrete and is applied to a prewetted surface.

(b) Latex-modified concrete requires a combination of wet and dry curing.

The air drying period after the initial moist cure allows the polymer particles to unite and bond. The air drying period is normally advantageous in field use of latex-modified concrete because it allows a bridge deck to be opened to traffic at an earlier date (20).

Mixture proportions of latex-modified concrete are different from those of low-slump concrete. The proportion of fine aggregate by the total weight is much higher and the cement content is about 650 lb/cu yd (384 kg/cu m). Although the water-cement ratio is low (0.35 to 0.40), the slump is high, typically 5 ±1 in (127 ±25 mm). Air entrainment is not required for resistance to freezing and thawing, and the air content, which is mainly entrapped air, is limited to 6.5% (1). However, it has been reported (20) that air content as high as 9% in fresh latex-modified concrete has no significant adverse effect on the properties of the material.

A FHWA study (20) has shown that latex-modified concretes exhibit slightly lower compressive strength and higher flexural strength than conventional concrete. Slant shear composite-compressive strength tests (an indirect measurement of bond) in this study showed that the bond between latex-modified concrete and underlying portland cement concrete is proportional to the compressive strength of the latex-modified concrete. The same study reported that concretes made with the latex emulsions were somewhat water-susceptible in that a reduction in compressive, flexural, and slant shear composite-compressive strengths occurred when the cured concretes were soaked in saturated limewater. It was also shown that the chloride permeability of latex-modified concrete was significantly less than that of conventional concretes having water/cement ratios of 0.40 by weight or more. However, the latex-modified concretes were not completely impermeable and thus would allow chloride ingress at a reduced rate.

Construction operations of latex-modified concrete are straightforward, however, installation of the material on decks with steep grades and crossfalls is difficult due to its high workability. Latex-modified concrete uses expensive material but requires less manpower than low-slump concrete and may be placed by conventional equipment (1).

High material costs and the superior performance of latex-modified concrete with respect to chloride permeability have led to latex-modified concrete overlays being thinner than most low-slump concrete overlays. Thicknesses of 1 1/4 to 1 1/2 in (32 to 38 mm) are reported. Performance has generally been satisfactory, though extensive cracking and some debonding have been reported, especially in overlays 3/4 in (19 mm) thick. Most of these cracks have been attributed to rapid initial shrinkage (1).

#### 4.1.2.1. Cost

The cost of latex-modified concrete depends upon the past development of the market in each area. In areas where more latexes have been introduced to the marketplace and strong licensees have been established, latex-modified concrete is more competitive than low-slump concrete (1). The cost of latex-modified concrete in the State of Washington for 1982 has been reported to be almost the same as dense-low slump concrete.

### 4.1.3. Internally Sealed Concrete Overlays

The process of making internally sealed concrete consists of mixing small discrete wax particles with the conventional components of portland cement concrete. After curing the concrete, heat is applied to melt and flow the wax into the capillaries and bleed channels of the concrete. Upon removal of the heat, the molten wax solidifies in the pores and the capillaries of concrete, and blocks the penetration of water and chloride (2).

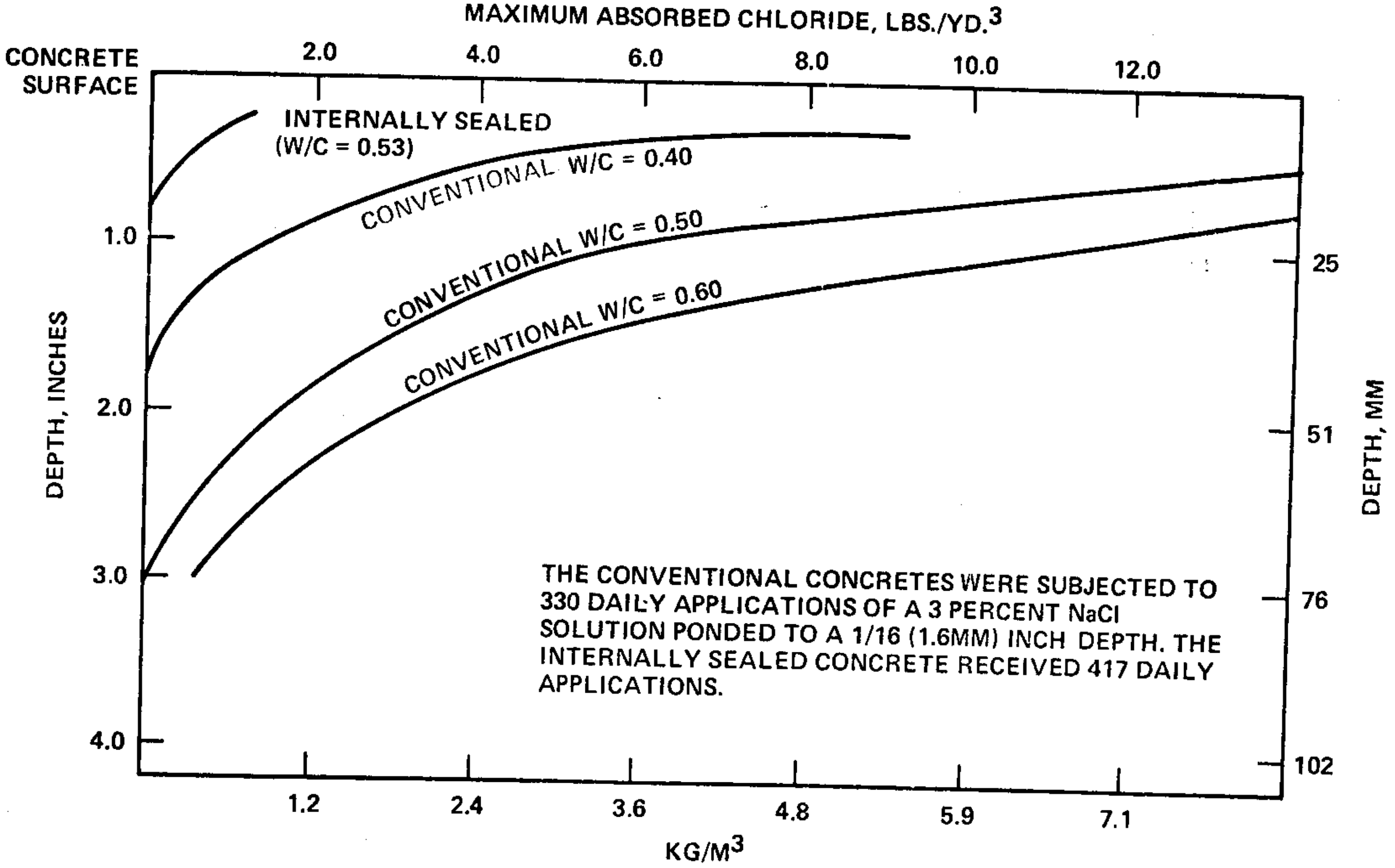
Research performed by the Monsanto Research Corporation has shown that concrete internally sealed with a blend of 25% montan wax and 35% paraffin wax (added to the fresh concrete as spherical, 20 to 80 mesh particles at 3% by weight of concrete) is completely resistant to chloride penetration (21). Chloride permeability of internally sealed concrete vs conventional concrete with different water/cement ratios is shown in Figure A-9. Laboratory tests have also indicated that internally sealed concrete is significantly less permeable than latex-modified concrete (20).

In FHWA internally sealed concrete studies (22) the compressive strength of internally sealed concrete was found to be decreased by the heating process. This loss, however, was of an acceptable amount and was recovered by the concrete as its aged increased. An "after heating" compressive strength of 400 psi (27.58 MPa) was attained when 14 day moist and 14 day air cure was used before heating. The same study (22) also demonstrated that non-air entrained internally sealed concrete was even more resistant to deicer scaling than conventional air-entrained concrete. The results of bond tests indicated that the bond of internally sealed concrete layer to an underlying hardened concrete placed with or without PC grout was satisfactory in which the heat treatment greatly enhanced the bond of the internally sealed overlay (22).

Mixture proportioning of internally sealed concrete is the same as for conventional concrete with an additional step of replacing an equal volume of sand or combination of sand and stone with 2.1 $ft^3$ of wax beads per cu yd (0.08 cu m/cu m) of concrete (23). The bead content will be approximately 3% by weight or 115 lb/cu yd (68 kg/cu m) of concrete (23). Due to the expansion of wax beads with temperature increases prior to melting, the beads should contain a void volume of at least 8% to avoid cracking the concrete on heating (22). The maximum allowable water-to-cement ratio should be 0.55 by weight, and slump must be between 2.5 and 4.0 (64 and 102 mm)(23).

Internally sealed concrete overlays are similar in handling, and placing characteristics to conventional concrete. Thicknesses of 2 or 3 in (51 or 76 mm) are usually used. Heat treatment is the final step in the construction process and can be accomplished by either a one-pass infrared heater or an electric blanket heating system. It is reported that (22) if the single pass heater is used, the concrete must be dried to a certain moisture level before heat treatment is begun, due to the pressures created by moisture trapped within the concrete which is a result of fast heating. On the other hand, the blanket system which covers a large area at once can be applied to a concrete of any moisture content due to its slow heating rate (22).

Research is in progress to develop an internally sealed concrete which does not require heating to achieve the sealing of concrete. The primary emphasis involves the use of encapsulation technology. The sealant material will be encapsulated inside a wall material which will slowly degrade in the



THE CONVENTIONAL CONCRETES WERE SUBJECTED TO 330 DAILY APPLICATIONS OF A 3 PERCENT NaCl SOLUTION PONDED TO A 1/16 (1.6MM) INCH DEPTH. THE INTERNALLY SEALED CONCRETE RECEIVED 417 DAILY APPLICATIONS.

Figure A-9. Chloride Penetration into Internally Sealed and Conventional Concretes (22) alkaline concrete environment, thus releasing the sealant after the concrete has cured (7).

#### 4.1.3.1. Cost

Internally sealed concrete is technically feasible, but its economic feasibility remains doubtful using present procedures. The cost of internally sealed concrete was reported to be \$3.79/ft$^2$ (\$40.80/ m$^2$ ) above that for a conventional concrete deck (24). Research is continuing in an attempt to improve the efficiency of the heat treatment process and also to identify different materials, which may eliminate the heat treatment entirely.

## 4.2 Waterproofing Membranes

When traffic volumes influence the choice of bridge deck repair systems, the use of expedient methods are required rather than those that are the most desirable. For example, it may be necessary to rehabilitate the deck in short sections using rapid-hardening patching materials followed immediately by an application of a waterproofing membrane and asphalt concrete wearing course (1).

### 4.2.1. Requirements for Membranes

Products of different materials and quality have been used since 1972 in the United States as waterproofing membranes. The potential performance of waterproofing systems under field condition have been reported in NCHRP Project "Waterproof Membranes for Protection of Concrete Bridge Decks" as follows (25):

(a) Form a complete and impervious barrier to penetration of water.

(b) Bridge, without rupture, those cracks that appear in the portland cement concrete bridge decks during the shrinkage of the concrete while curing, and periodically change in width with thermal or moisture cycle.

(c) Form a sufficient bond between the membrane system and the portland cement concrete deck to resist slippage during construction and under traffic.

(d) Withstand without deterioration the elevated temperature resulting from placement of a hot asphaltic concrete wearing course on the membrane system.

(e) Resist the normal action of construction equipment during placement of an asphaltic concrete wearing course over the membrane system.

(f) Resist the slow penetration of the asphaltic concrete aggregate (creep damage) resulting from traffic on the wearing course.

### 4.2.2. Applied-in-Place vs Preformed Sheet Systems

Generally, waterproofing membrane systems are classified as preformed sheet systems and liquid or applied-in-place materials. The major differences between these two groups of products are presented in Table A-4 (1). Laboratory investigations have indicated the superiority of preformed membranes, which may be expected to perform better than applied-in-place materials in simple screening tests (25).

### 4.2.3. Protective Coverings

An asphaltic concrete wearing course is used on almost all waterproofing systems to provide a riding surface of the deck. The use of a separate wearing course, however, can dictate the economic life of the membrane if it is not possible to replace the existing wearing course with the membrane intact. Some products also require the use of a protective system between the membrane and the wearing course to prevent damage from the installation of hot mix and to resist the penetration of the asphaltic concrete aggregate under service conditions. In this case, asphalt-impregnated boards, typically 1/8 in (3 mm), are usually used as protective layers (1).

### 4.2.4. Blistering

Blistering affects both preformed sheet and applied-in-place materials. Blisters may be caused by the expansion of air in the concrete, water vapor in the concrete or vaporization of solvents used in prime coats and adhesives after application of the membrane (1). Blisters may take several forms ranging from numerous pinholes to single blisters that may cover a square foot or more. One solution to the blistering problem is to apply the membrane when the deck temperature is higher than ambient temperature. Blisters may also be formed by the rapid expension of vapor during placement of the hot mix which, in turn, may cause cracking of the asphalt. In this case, a 2 in (51 mm) thick asphalt wearing course has been reported to have sufficient dead load to prevent blistering, though a 3 in (76 mm) thickness is preferable (1). An alternative method of preventing blisters is to allow the vapor pressure to disperse through a venting layer beneath the membrane, such as a perforated sheet of bituminous felt (1).

Table A-4.

Characteristics of Membranes (1)

| Applied-in-Place Materials | Preformed Systems |
|---|---|
| Difficult to assure the quality of two-component materials and products which are hot applied. | Quality of material controlled under factory conditions. |
| Careful field inspection required to control thickness of membrane and detect presence of pinholes. | Thickness and integrity controlled at the factory. |
| Usually applied in one course by spray or squeegee. No laps required. | Labor-intensive installation, especially if not self-adhesive. Laps necessary. |
| Application independent of deck geometry. | Difficult to install on curved or rough decks. |
| Bonding not usually a problem if substrate properly prepared. Systems usually self-adhesive. | Cured sheets may be difficult to bond to substrate, protection layer and at laps. |
| Installation not affected by deck details. | Vulnerable to quality of workmanship at critical locations such as curbs, expansion joints and deck drains. |
| Blisters and blowholes easily repaired in self-sealing materials. | Blisters must be repaired by puncturing and patching. |
| Tend to be less expensive. | Tend to be more expensive. |

### 4.2.5. Field Performance of Membranes

Because of the tendency of membranes to slip, they should not be used on grades greater than 4% or in areas subject to rapid acceleration, deceleration, or turning movements. The field performance of membranes has been reported (25) to be highly variable. Field surveys and measurements have indicated that none of the membrane systems in service are performing their function in a completely satisfactory manner. The major reasons for this generally unsatisfactory performance have been reported in the NCHRP Project (25) as follows:

(a) Use of systems that are not impermeable because of defects, such as pinholes or bubbles in the film, or tendency to emulsify in contact with water.

(b) Improper construction practices, resulting in damage to the membrane during placement of the asphaltic concrete wearing course over the membrane.

(c) Inability of membrane systems under service conditions to extend sufficiently to bridge moving cracks in the concrete deck, especially at low temperatures.

(d) Membrane systems that softened excessively during high ambient temperature service, resulting in lateral movement in the wheelpaths or upward migration into the asphaltic concrete wearing course, thereby leaving insufficient material to function as a membrane.

The electrical resistance test can be used for assessment of water permeability of membrane systems (see Appendix B). The electrical resistance of a membrane is a measure of its impermeability. If a membrane does not perform well after it is placed and water can pass through it, the electrical resistance will be low. On the other hand, effective waterproofing membranes provide very high resistance when placed. Spellman and Stratfull (26) have suggested that an excellent waterproofing material would have an electrical resistance greater than 500 $K\Omega/ft^2$ (500 $K\Omega/0.09\ m^2$), a poor material would have a resistance less than 100 $K\Omega/ft^2$ (100 $K\Omega/0.09\ m^2$), and the performance of waterproofing materials having resistance values between these two are questionable.

### 4.2.6 Cost

The current cost (1982) of installation of a protective membrane is approximately \$0.90/ft$^2$ (\$9.68/m$^2$) and that of an asphaltic concrete overlay is about \$0.90/ft$^2$ (\$9.68/m$^2$) (2). Thus, the total cost of installation of the protective system will be aoubt \$1.80/ft$^2$ (\$19.36/m$^2$).

## 4.3 Cathodic Protection

Cathodic protection has been used since the mid-1930s to prevent steel corrosion in pipelines. However, the application of cathodic protection to bridge decks is relatively new and presents some problems not encountered with baried structures such as the lack of a suitable conductive environment for the anode (1).

The basic hypothesis of cathodic protection is that all galvanic corrosion on a structure is halted when all points on its surface are polarized to a potential at least equal to the open-circuit potential of the most anodic point on the structure (5). In this condition corroding anodes on the steel are prevented from discharging ions. Thus, the current discharging anodes become current-receiving cathodes; hence the term cathodic protection (1).

For steel pipelines, an empirical criterion is that the steel must be made more negative than -0.35 volts CSE (relative to the copper/copper-sulfate half-cell, CSE). The most anodic half-cell potentials of corroded steel in corrosion-cracked concrete measured by Stratfull (12) was -0.67 volts CSE. Stratfull has recommended that for the satisfactory cathodic protection of steel in concrete, the potentials of the steel should be no less than -0.85 volts CSE and no more than -1.10 volts CSE to avoid the possibility of loss of bond strength.

### 4.3.1. Methods of Cathodic Protection

The cathodic protection of metal is applied by transmitting a flow of current from any source to the metal. Two methods to transmit the protective current are: (a) galvanic anodes (sacrificial anodes), and (b) impressed current.

In the galvanic anode system, a sacrificial metal that is anodic to the metal of the structure (i.e., higher in electromotive series) is connected to the metal of the structure. Thus, a current will be provided naturally causing the metal under protection to receive current and become a cathode (28), as shown in Figure A-10. The anode metal in this case is sacrified since, the corrosion is transferred from the metal of the structure to the anode metal, which is expendable and may be easily replaced. The driving potential for the current is the natural potential difference between the metal of the structure and the anode. Some typical potential relationships of metals are shown in Table A-5. As shown in the table , magnesium offers cathodic protection to any of the other metals listed, whereas copper does not. For the cathodic protection of steel reinforcement in concrete, mainly magnesium and zinc are used (5).

The galvanic anode system has two important drawbacks, which are: (a) voltage between metals is limited to the maximum electrical potential difference, and (b) current output of the galvanic anode will change as the electrical resistance of the electrolyte changes (28). This means that, because concrete has a relatively high resistivity, numerous anodes are required. The galvanic anode system, on the other hand, when properly installed, may be expected to operate continually without maintenance for the life of the structure. Because the driving voltage is inherent and the current and voltages involved are relatively small, there is little chance of system failure (5).

The impressed current approach to cathodic protection operates in the same manner as the galvanic anode system, expect that the current into the structure receiving protection is provided by an external power (see Figure A-11). Thus, the current flow does not depend on the relative potentials of the anode metal and the metal of the structure. This permits the anode to be selected on the basis of its capability to conduct current with a minimum amount of corrosion to the anode. Carbon and high-silicon cast iron have been used most often in the impressed current system. The protective current for impressed current systems may be obtained from batteries. However, the most common source is the rectifier, which converts alternating current power to direct current power (5).

For bridge decks, the impressed current system has a number of distinct advantages, which are (a) voltage output of the anodes can be varied from less than one volt to over 100 volts, and (b) current output can be automatically controlled regardless of electrical resistance of the electrolyte (28). However, the use of high current and voltage may result in possible overprotection of reinforced concrete structures, which may cause an adverse effect on

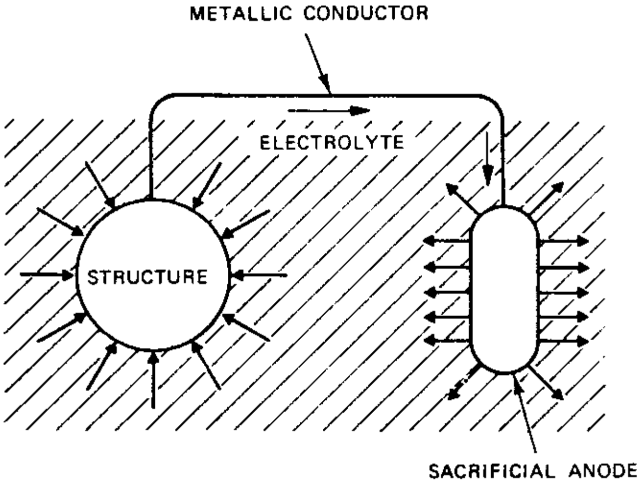


Figure A-10. Sacrificial Anode Cathodic Protection (5)

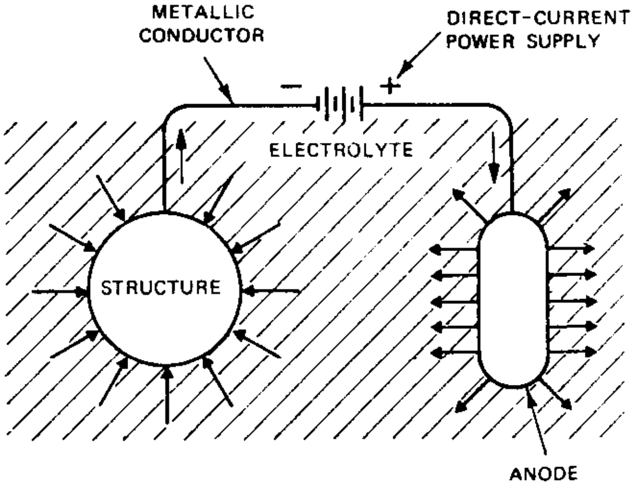


Figure A-11. Impressed Current Cathodic Protection (5)

## Table A-5.

## Typical Potential Relationship of Metals Measured to the Copper Sulfate Reference (5)

| *Metal* | *Typical Potential, volts*[a] |
|---|---|
| Magnesium alloy (6% Al, 3% Zn, and 0.15% Mn) | −1.60 |
| Zinc | −1.1 |
| Aluminum alloy (5% Zn) | −1.05 |
| Mild steel (rusted) | −0.2 to −0.5 |
| Mild steel in concrete | −0.2 |
| Copper | −0.2 |

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[a] Typical potential normally observed in neutral soils and water, measured to the copper sulfate reference.

the bond between the concrete and the steel. Laboratory studies have shown that hydrogen gas bubbles began to form at the cathodic steel surface at -1.17 volts CSE in high pH solutions (5). Although the passage of an electrical current through concrete does not have an adverse effect on the concrete itself, hydrogen released at the cathodic steel surface may soften the concrete and decrease the bond between the concrete and the steel. Softening of the concrete near the cathode was first demonstrated by National Bureau of Standards tests published in 1913 (1, 29).

### 4.3.2 Conductive Layer

Although methods for applying cathodic protection to structures, such as buried pipelines and concrete water tanks, are well-established, a different system for applying cathodic protection is required for a bridge deck. In the case of bridge decks, there is no surrounding conducting medium since the deck is suspended in air. If the concrete of deck was used as a conducting medium and anodes were placed in the concrete, only reinforcing steel bar close to the anodes theoretically could be protected. Thus, in order to equally protect all the reinforcing steel in the deck numerous anodes would have been required (28). The assumed electrical values for this circuit are shown in Figure A-12 (28). To reduce the number of anodes and at the same time get current effectively to the reinforcing steel, Stratfull showed that the anodes must be removed from the high resistivity concrete and placed in a electrically conductive overlay on the concrete surface to provide equal resistance from all bars to the anodes as shown in Figure A-13 (28).

Stratfull (28) solved this problem by using a conductive layer consisting of a coke breeze and asphalt mixture on the deck. The coke mix was laid and energized by anodes at several points so that an equal distribution of power was possible over the deck. The coke breeze mix was covered with a wearing course of asphalt concrete to provide a riding surface of the deck. The circuit diagram for such an installation is shown in Figure A-14. The coke breeze mixture consisted of 85% coke aggregate of 3/8 in (10 mm) maximum size and 15% 85-100 penetration asphalt which was followed by a 2 in (51 mm) thick conventional wearing course for a total overlay thickness of 5 in (127 mm) over the deck.

The mixture of asphalt and coke breeze had a high void content and a low stability. Therefore, it had the danger of rutting under heavy traffic and

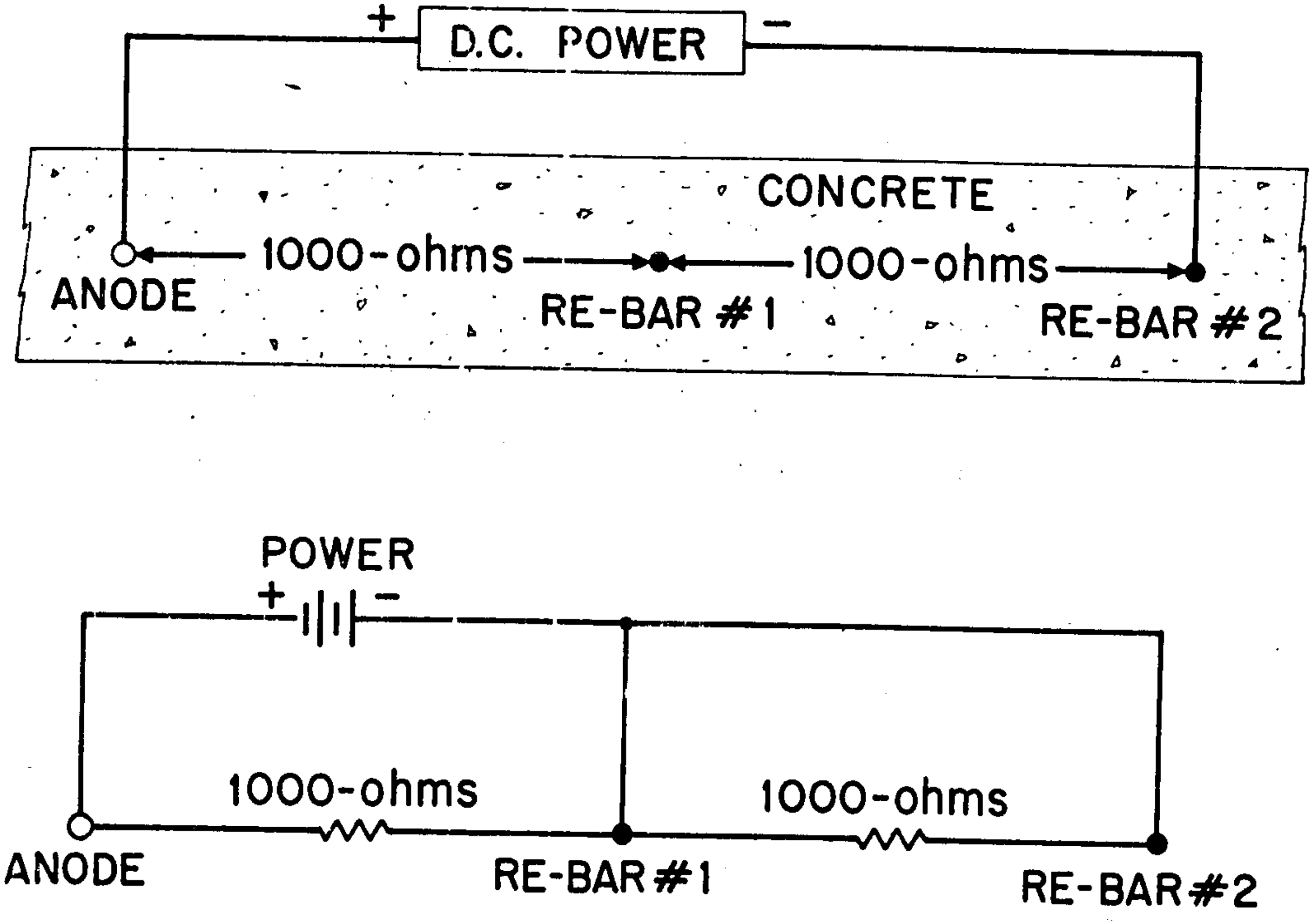

Figure A-12. Schematic of Anode in Concrete (28)

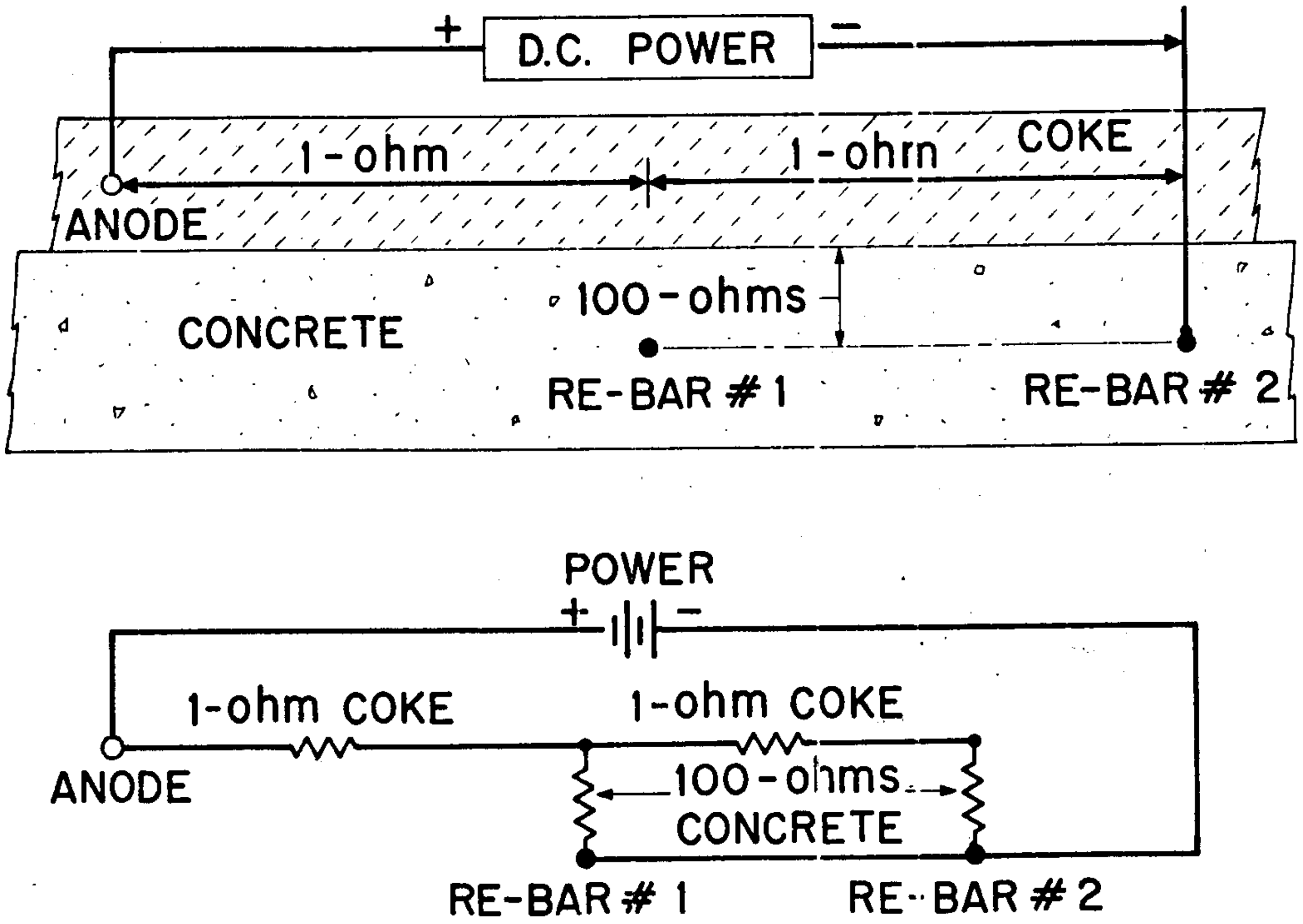

Figure A-13. Schematic of Anode in Coke (28)

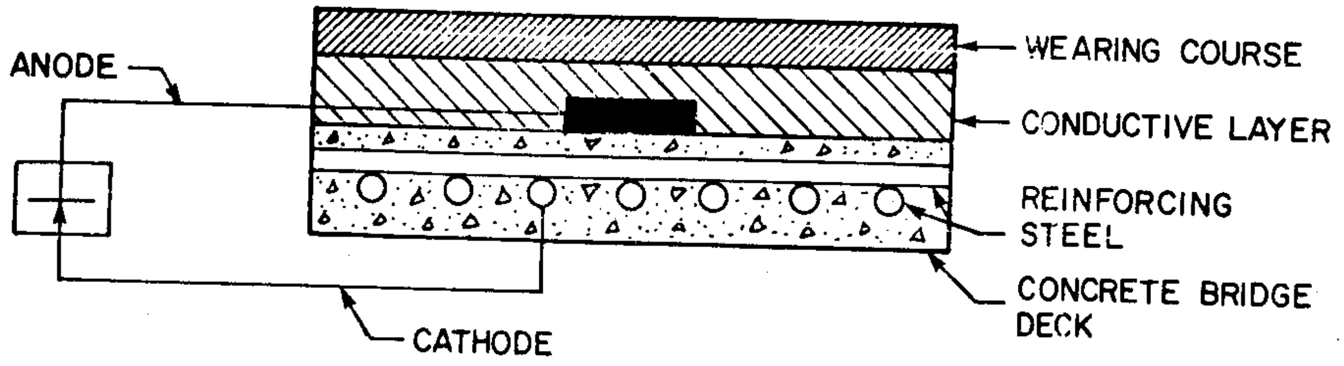

Figure A-14. Impressed Current Cathodic Protection Circuit (1) stripping of the asphalt caused by brine accumulation in the pores. This could also accelerate deterioration of underlying concrete. Later it was found that by incorporating fine and coarse aggregates into the mixture of asphalt and coke breeze, a conductive mixture could be made having structural characteristics almost comparable to conventional asphalt mixtures. The mixture proportions used were stone/sand/coke breeze in the ratios of 40/15/45, with an asphalt content of 15.5% by weight of the total aggregate. Such a mixture achieved a resistivity of 1.18 ohm-in (3.0 ohm-cm) compared with 0.55 ohm-in (1.4 ohm-cm) for a mixture of 80% coke breeze and 20% asphalt (1).

In installation of conductive layers it is important that the coke mixture be insulated from any bare reinforcing steel, deck scuppers, and expansion joints to prevent a direct short circuit to the bride steel (32), which can negate the system.

The long-term effects of the conductive mixture on the concrete deck slab are not known. Since a membrane, which is a dielectric material, cannot be used to protect the concrete against freezing and thawing, the concrete should be sound and properly air-entrained (1).

Recently a NCHRP research effort concentrated on the development of conductive portland cement mortars and on the use of conductive paints in impressed current cathodic protection of concrete bridge substructures (31). Two conductive systems were evaluated for use as a secondary anode. One consisted of a portland cement mortar made conductive by use of a graphite pigment that is electrochemically inert. A typical conductive portland cement mortar exhibited a resistivity of 31.50 ohm-in (80 ohm-cm) and a 28-day compressive strength of 2890 psi (19.98 MPa). Use of this material on test slabs was reported only partially successful with shrinkage, disbondment during thermal cycling and freezing and thawing. The high absorption characteristics of the carbon fillers and aggregates result in a very high water cement ratio concrete with poor material strength and durability (7).

Work with conductive paints, however, has been reported (7, 31) much more promising. A total of 9 commercially available conductive paints based on electronic, nonmetallic, non-ionic conductive mechanisms were evaluated in the NCHRP studies (31) for use as a secondary anode for a impressed current system. Resistivity tests indicated that a 2 mil (51 $\mu$) thickness of some conductive paints would be equivalent to a 1-in (25-mm) thick section of 31.50 ohm-in (80 ohm-cm) conductive mortar. Thermal cycling and freeze-thaw testing did not cause any deterioration of the material. The conductive paint system will require applications of 6 to 10 mils (152 to 254 μ) dry film thickness to achieve optimum spacing between primary anode attachements. It is reported that the material could be applied with conventional air spray to wet film thickness of 20 mils (508 μ) on vertical surfaces (7, 31).

#### 4.3.2.1. Types of Anode

Two types of impressed current anodes are in general use: graphite and a silocon-iron alloy composition. The sue of disc-shaped, 10 in (254 mm) diameter, 1 1/4 in (32 mm) thick, iron alloy anodes has been reported in conjunction with asphalt coke breeze conductive layers (28). In this case the iron alloy anodes were fastened to the deck using an epoxy adhesive. The purpose of the adhesive was (1): (a) to secure the anodes during paving, and (b) to prevent creating localized areas of high potential caused by direct current flow from the anode to the steel.

One of the most significant advances in cathodic protection has been the installation of anodes such that future replacement if wearing course and conductive layer won't damage the electrical circuitry. In this case, anodes are recessed in the concrete deck surface. The wires for the hardware are also placed in saw cuts in the deck surface (1).

More recently, it has been shown that when using conductive coatings on substructures as secondary anodes, the size and shape of primary anodes must be such that permanent contact with the conductive coating can be accomplished easily. For example, thin wires, plates or expanded plates are favored over bulky, thick and wide shapes (31).

### 4.3.3. Non-Overlay Systems

Another significant recent development in the bridge deck cathodic protection area is the non-overlay systems proposed by private industry (7). The entire system, which consisted of a platinized niobium, copper-core wire, was placed in slots cut in the deck surface. The saw slots were then grouted with a very low resistivity grout which was expected to perform adequately on a bridge deck riding surface. Although the anode spacing is smaller than that of the overlay system, the predicted cost of the non-overlay system is less than that of a coke-asphalt conductive overlay system. However, FHWA studies, reported earlier (33), indicated that only a very small spacing is possible and acid attack problems must be overcome. Recently a conductive polymer has been developed in FHWA laboratories and used with non-overlay systems and the results of this work have been reported very promising.

### 4.3.4. Field Performance of Cathodic Protection

The success of cathodic protection for steel reinforcement in conrete requires electrical continuity of the deck rebar (5, 28). If one segment of the reinforcement is electrically isolated from the remainder, it may not be protected cathodically and even could be damaged by cathodic interference (a form of stray-current corrosion in which the offending current is from a cathodic protection system) (5).

If the stray-current corrosion occurs, the resultant concrete spalling may be no different from the condition that exists before cathodic protection is applied. However, if such a damage occurs, repair can be made and the isolated segments of steel can be welded to adjacent reinforcing steel. They then become a part of the protected grid (28).

The use of cathodic protection for bridge decks, thus, relates to the electrical continuity of the deck rebars. Recent studies by Stratfull indicate that the electrical continuity may prove to be satisfactory (5).

Prior to applying cathodic protection, delaminations should be repaired by replacing all unsound concrete (1).

The practical application of a cathodic protection requires a field measurement of the polarized potential of the structure by using a reference electrode to maintain the potential within the required range in order to prevent over and under protection. The reference electrode can be placed at some point within the electrical field of cathodic protection. The measured potential value includes, in addition to the polarized potential of the structure, the potential gradient caused by the flow of protective current through the resistance of the electrolyte spanned by structure to the reference point. The concrete-pipeline procedure, to eliminate the potential gradient, is to momentarily interupt the flow of protective current in order to make the potential measurement the instant the current ceases to flow. By using this procedure, the location of the reference electrode does not affect the measured potential values, because the potential gradient no longer exists (5).

The polarization of the steel is maintained for several days after the power is switched off. This confirms that the system is not susceptible to sudden loss of power if, for instance, the control panel or rectifier is damaged by lightning or vandalism (1).

### 4.3.5. Cost

Cathodic protection is the only method, short of removal of all chloride contaminated concrete, that is sure to stop active corrosion in a bridge

deck (1). The equipment used with the impressed current method requires periodic monitoring, which does not satisfy the requirement for a completely maintenance-free deck. However, when the galvanic anode method is used, it may be expected to operate continually without maintenance for the life of the anodes.

The cost of cathodic protection is highly variable depending on the local availability of the coke breeze and the AC power supply. Figures in the range of \$11.00 to \$17.00/ft$^2$ (\$118.00 to \$183/m$^2$) have been reported for 1982.

# APPENDIX B

## SELECTED ASTM AND AASHTO PROCEDURES

AMERICAN NATIONAL STANDARD ANSI/ASTM C 876 – 77

# Standard Test Method for
# HALF CELL POTENTIALS OF REINFORCING STEEL IN CONCRETE[1]

This Standard is issued under the fixed designation C 876; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

## 1. Scope

1.1 This method covers the estimation of the electrical half cell potential of reinforcing steel in concrete, for the purpose of determining the corrosion activity of the reinforcing steel. This method may be used advantageously to assess the relative extent and progress of corrosion of steel embedded in field and laboratory concrete members. This method is applicable to members regardless of their size or the depth of concrete cover over the reinforcing steel.

## 2. Significance and Use

2.1 The results obtained by the use of this method should not be considered as a means for estimating the structural properties of the steel or of the reinforced concrete member.

2.2 This method is limited by electrical circuitry. A concrete surface that has dried to the extent that it is a dielectric and surfaces that are coated with a dielectric material will not provide an acceptable electrical circuit. The basic configuration of the electrical circuit is shown in Fig. 1.

## 3. Apparatus

3.1 The testing apparatus consists of the following:

3.1.1 *Half Cell:*

3.1.1.1 A copper-copper sulfate half cell (Note 1) is shown in Fig. 2. It consists of a rigid tube or container composed of a dielectric material that is nonreactive with copper or copper sulfate, a porous wooden or plastic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulfate. The solution shall be prepared with reagent grade copper sulfate crystals dissolved in distilled or deionized water. The solution may be considered saturated when an excess of crystals (undissolved) lies at the bottom of the solution.

3.1.1.2 The rigid tube or container shall have an inside diameter of not less than 1 in. (25 mm); the diameter of the porous plug shall not be less than $^1/_2$ in. (13 mm); the diameter of the immersed copper rod shall not be less than $^1/_4$ in. (6 mm), and the length shall not be less than 2 in. (50 mm).

3.1.1.3 Present criteria based upon the half cell reaction of $Cu \rightarrow Cu^{++} + 2e$ indicate that the potential of the saturated copper-copper sulfate half cell as referenced to the hydrogen electrode is −0.316 V at 72°F (22.2°C). The cell has a temperature coefficient of about 0.0005 V more negative per °F for the temperature range from 32 to 120°F (0 to 49°C).

NOTE 1—While this method specifies only one type of half cell, that is, the copper-copper sulfate half cell, others having similar measurement range, accuracy, and precision characteristics may also be used. In addition to copper-copper sulfate cells, calomel cells have been used in laboratory studies. Potentials measured by other than copper-copper sulfate half cells should be converted to the copper-copper sulfate equivalent potential. The conversion technique is described in most physical chemistry or half cell technology text books.

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[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03.15 on Methods of Testing Concrete for Resistance to Weathering.

Current edition approved Dec. 30, 1977. Published February 1978.

3.1.2 *Electrical Junction Device* — An electrical junction device shall be used to provide a low electrical resistance liquid bridge between the surface of the concrete and the half cell. It shall consist of a sponge or several sponges pre-wetted with a low electrical resistance contact solution. The sponge may be folded around and attached to the tip of the half cell so that it provides electrical continuity between the porous plug and the concrete member.

3.1.3 *Electrical Contact Solution* — In order to standardize the potential drop through the concrete portion of the circuit, an electrical contact solution shall be used to wet the electrical junction device. One such solution is composed of a mixture of 95 ml of wetting agent (commercially available wetting agent) or a liquid household detergent thoroughly mixed with 5 gal (19 litre) of potable water. Under working temperatures of less than about 50°F (10°C), approximately 15 % by volume of either isopropyl or denatured alcohol must be added to prevent clouding of the electrical contact solution, since clouding may inhibit penetration of water into the concrete to be tested.

3.1.4 *Voltmeter* — The voltmeter shall have the capacity of being battery operated and have ±3 % end-of-scale accuracy at the voltage ranges in use. The input impedance shall be no less than 10 million Ω when operated at a full scale of 100 mV. The divisions on the scale used shall be such that a potential difference of 0.02 V or less can be read without interpolation.

3.1.5 *Electrical Lead Wires* — The electrical lead wire shall be of such dimension that its electrical resistance for the length used will not disturb the electrical circuit by more than 0.0001 V. This has been accomplished by using no more than a total of 500 linear ft (150 m) of at least AWG No. 24 wire. The wire shall be suitably coated with direct burial type of insulation.

## 4. Procedure

4.1 *Spacing Between Measurements* — While there is no pre-defined minimum spacing between measurements on the surface of the concrete member, it is of little value to take two measurements from virtually the same point. Conversely, measurements taken with very wide spacing may neither detect corrosion activity that is present or result in the appropriate accumulation of data for evaluation. The spacing shall therefore be consistent with the member being investigated and the intended end use of the measurements (Note 2).

NOTE 2 — A spacing of 4 ft (1.2 m) has been found satisfactory for evaluation of bridge decks. Generally, larger spacings increase the probability that localized corrosion areas will not be detected. Measurements may be taken in either a grid or a random pattern. Spacing between measurements should generally be reduced where adjacent readings exhibit algebraic reading differences exceeding 150 mV (areas of high corrosion activity). Minimum spacing generally should provide at least a 100-mV difference between readings.

4.2 *Electrical Connection to the Steel:*

4.2.1 Make a direct electrical connection to the reinforcing steel by means of a compression-type ground clamp, or by brazing or welding a protruding rod. To ensure a low electrical resistance connection, scrape the bar or brush the wire before connecting to the reinforcing steel. In certain cases, this technique may require removal of some concrete to expose the reinforcing steel. Connect the electrical connection to the reinforcing steel to the negative terminal of the voltmeter.

4.2.2 Attachment must be made directly to the reinforcing steel except in cases where it can be documented that an exposed steel member is directly attached to the reinforcing steel. Certain members, such as expansion dams, date plates, lift works, and parapet rails may not be attached directly to the reinforcing steel and, therefore, may yield invalid readings. Electrical continuity of steel components with the reinforcing steel can be established by measuring the resistance between widely separated steel components on the deck. Where duplicate test measurements are continued over a long period of time, identical connection points should be used each time for a given measurement.

4.3 *Electrical Connection to the Half Cell* — Electrically connect one end of the lead wire to the half cell and the other end of this same lead wire to the positive terminal of the voltmeter.

4.4 *Pre-Wetting of the Concrete Surface:*

4.4.1 Under certain conditions, the concrete surface or an overlaying material, or both, must be pre-wetted by either of the two methods described in 4.4.3 or 4.4.4 with the solution described in 3.1.2 to decrease the electrical resistance of the circuit.

4.4.2 A test to determine the need for pre-wetting may be made as follows:

4.4.2.1 Place the half cell on the concrete surface and do not move.

4.4.2.2 Observe the voltmeter for one of the following conditions:

(*a*) The measured value of the half cell potential does not change or fluctuate with time.

(*b*) The measured value of the half cell potential changes or fluctuates with time.

4.4.2.3 If condition (*a*) is observed, pre-wetting the concrete surface is not necessary. However, if condition (*b*) is observed, pre-wetting is required for an amount of time such that the needle voltage reading is constant when observed for at least 5 min. If pre-wetting cannot obtain condition (*a*), either the electrical resistance of the circuit is too great to obtain valid half cell potential measurements of the steel, or stray current from a nearby direct current traction system or other fluctuating direct current, such as arc welding, is affecting the readings. In either case, the half cell method should not be used.

4.4.3 *Method A for Pre-Wetting Concrete Surfaces* — Use Method A for those conditions where a minimal amount of pre-wetting is required to obtain condition (*a*) as described in 4.4.2.2. Accomplish this by spraying or otherwise wetting either the entire concrete surface or only the points of measurement as described in 4.1 with the solution described in 3.1.3. No free surface water should remain between grid points when potential measurements are initiated.

4.4.4 *Method B for Pre-Wetting Concrete Surfaces* — In this method, saturate sponges with the solution described in 3.1.3 and place on the concrete surface at locations described in 4.1. Leave the sponges in place for the period of time necessary to obtain condition (*a*) described in 4.4.2.2. Do not remove the sponges from the concrete surface until after the half cell potential reading is made. In making the half cell potential measurements, place the electrical junction device described in 3.1.2 firmly on top of the pre-wetting sponges for the duration of the measurement.

4.5 *Underwater, Horizontal, and Vertical Measurements:*

4.5.1 Potential measurements detect corrosion activity, but not necessarily the location of corrosion activity. The precise location of corrosion activity requires knowledge of the electrical resistance of the material between the half cell and the corroding steel. While underwater measurements are possible, results regarding the location of corrosion must be interpreted very carefully. Often it is not possible to precisely locate points of underwater corrosion activity in salt water environments because potential readings along the member appear uniform. However, the magnitude of readings does serve to indicate whether or not active corrosion is occurring. Take care during all underwater measurements that the half cell does not become contaminated and that no part other than the porous tip of the copper-copper sulfate electrode half cell comes in contact with water.

4.5.2 Perform horizontal and vertically upward measurements exactly as vertically downward measurements. However, additionally ensure that the copper-copper sulfate solution in the half cell makes simultaneous electrical contact with the porous plug and the copper rod at all times.

4.6 *Care of the Half Cell* — The porous plug should be covered when not in use for long periods to ensure that it does not become dried to the point that it becomes a dielectric (upon drying, pores may become occluded with crystalline copper-sulfate). If cells do not produce the reproducibility or agreement between cells described in 3.1.1, cleaning the copper rod in the half cell may rectify the problem. The rod may be cleaned by wiping it with a dilute solution of hydrochloric acid. The copper sulfate solution should be renewed either monthly or before each use, whichever is the longer period. At no time should steel wool or any other contaminant be used to clean the copper rod or half cell tube.

## 5. Recording Half Cell Potential Values

5.1 Record the electrical half cell potentials to the nearest 0.01 V. By convention, a negative (−) sign is used for all readings. Report all half cell potential values in volts and correct for temperature if the half cell temperature is outside the range of 72 ± 10°F (22.2 ± 5.5°C). The temperature coefficient for the correction is given in 3.1.1.3.

## 6. Data Presentation

6.1 Test measurements may be presented by one or both of two methods. The first, an equipotential contour map, provides a graphical delineation of areas in the member where corrosion activity may be occurring. The second method, the cumulative frequency diagram, provides an indication of the magnitude of affected area of the concrete member.

6.1.1 *Equipotential Contour Map*—On a suitably scaled plan view of the concrete member, plot the locations of the half cell potential values of the steel in concrete and draw contours of equal potential through points of equal or interpolated equal values. The maximum contour interval shall be 0.10 V. An example is shown in Fig. 3.

6.1.2 *Cumulative Frequency Distribution*—To determine the distribution of the measured half cell potentials for the concrete member, make a plot of the data on normal probability paper in the following manner:

6.1.2.1 Arrange and consecutively number all half cell potentials by ranking from least negative potential to greatest negative potential.

6.1.2.2 Determine the plotting position of each numbered half cell potential in accordance with the following equation:

$$f_x = \frac{r}{\Sigma n + 1} \times 100$$

where:

$f_x$ = plotting position of total observations for the observed value, %,

$r$ = rank of individual half cell potential, and

$\Sigma n$ = total number of observations.

6.1.2.3 Label the ordinate of the probability paper "Half Cell Potential (Volts, CSE)," where CSE is the designation for copper-copper sulfate electrode. Label the abscissa of the probability paper "Cumulative Frequency (%)." Draw two horizontal parallel lines intersecting the −0.20 and −0.35 V values on the ordinate, respectively, across the chart.

6.1.2.4 After plotting the half cell potentials, draw a line of best fit through the value (Note 3). An example of a completed plot is shown in Fig. 4.

NOTE 3—It is not unusual to observe a break in the straight line. In these cases, the line of best fit shall be two straight lines that intersect at an angle.

## 7. Interpretation of Results[3]

7.1 Laboratory testing of reinforced concrete specimens indicates the following regarding the significance of the numerical value of the potentials measured. Voltages listed are referenced to the copper-copper sulfate (CSE) half cell.

7.1.1 If potentials over an area are numerically less than −0.20 V CSE, there is a greater than 90 % probability that no reinforcing steel corrosion is occurring in that area at the time of measurement.

7.1.2 If potentials over an area are in the range of −0.20 to −0.35 V CSE, corrosion activity of the reinforcing steel in that area is uncertain.

7.1.3 If potentials over an area are numerically greater than −0.35 V CSE, there is a greater than 90 % probability that reinforcing steel corrosion is occurring in that area at the time of measurement.

7.1.4 In laboratory tests where potentials were numerically greater than −0.50 V, approximately half of the specimens cracked due to corrosion activity.

7.1.5 Positive readings, if obtained, generally indicate insufficient moisture in the concrete and should not be considered valid.

## 8. Report

8.1 The report shall include the following:

8.1.1 Type of cell used if other than copper-copper sulfate.

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[3] The following published reports give supportive detail for interpretation of results: Spellman, D. L., and Stratfull, R. F., "Concrete Variables and Corrosion Testing," *Highway Research Record No. 433*, 1973; Stratfull, R. F., "Half-Cell Potentials and the Corrosion of Steel in Concrete," *Highway Research Record No. 423*, 1973; and Clear, K. C., and Hay, R. E., "Time to Corrosion of Reinforcing Steel in Concrete Slabs," Federal Highway Administration, Vol 1 and 2, *Interim Reports FHWA-RD-73-32 and 33*, April 1973.

8.1.2 The estimated average temperature of the half cell during the test.

8.1.3 The method for pre-wetting the concrete member and the method of attaching the voltmeter lead to the reinforcing steel.

8.1.4 An equipotential contour map, showing the location of reinforcing steel contact, or a plot of the cumulative frequency distribution of the half cell potentials, or both.

8.1.5 The percentage of the total half cell potentials that are more negative than −0.35 V.

8.1.6 The percentage of the total half cell potentials that are less negative than −0.20 V.

## 9. Precision

9.1 The difference between two half cell readings taken at the same location with the same cell should not exceed 10 mV when the cell is disconnected and reconnected.

9.2 The difference between two half cell readings taken at the same location with two different cells should not exceed 20 mV.

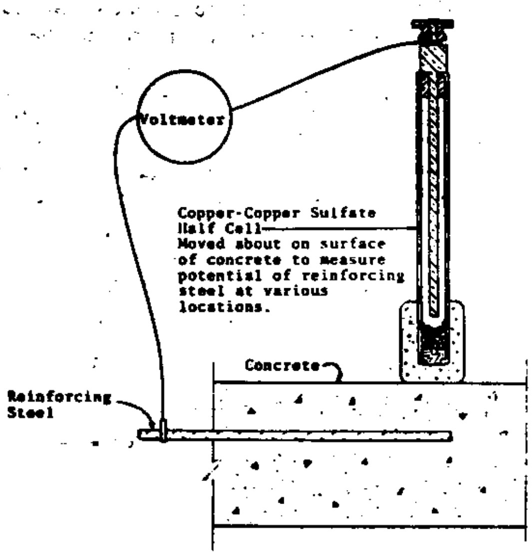

FIG. 1 Copper-Copper Sulfate Half Cell Circuitry.

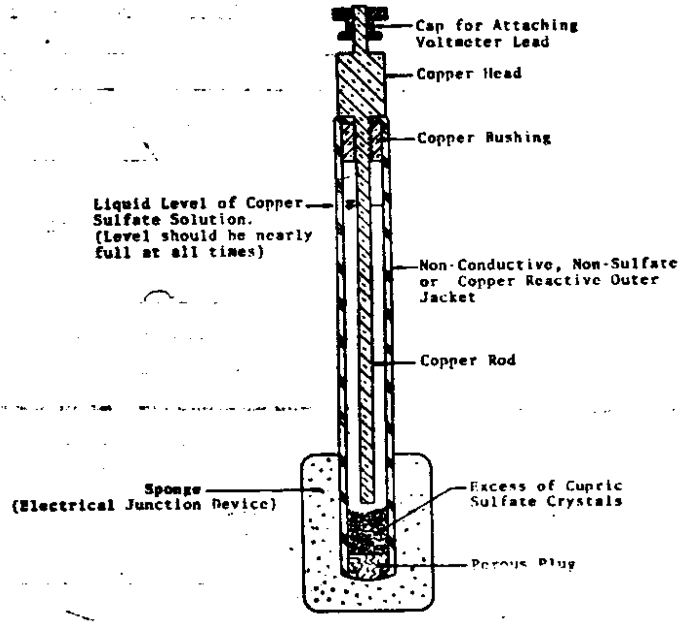

**FIG. 2 Sectional View of a Copper–Copper Sulfate Half Cell.**

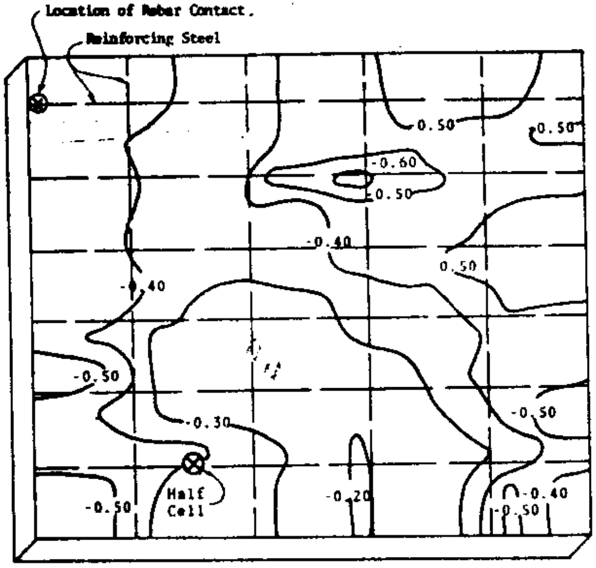

**FIG. 3 Equipotential Contour Map.**

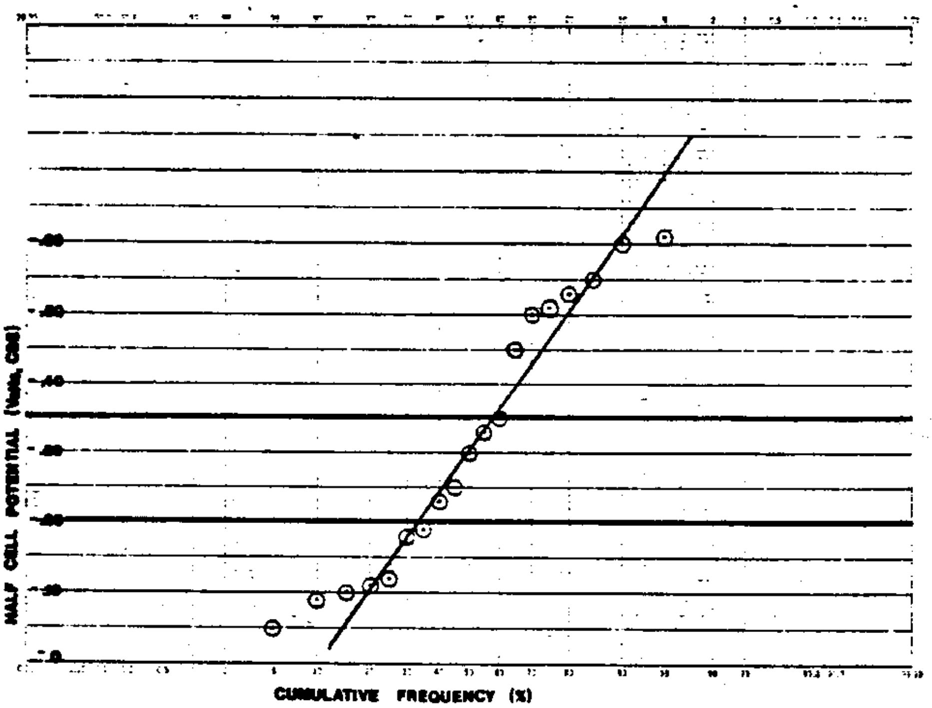

FIG. 4 Cumulative Frequency Diagram.

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AMERICAN NATIONAL STANDARD ANSI/ASTM D 3633 - 77

# Standard Test Method for
# ELECTRICAL RESISTIVITY OF MEMBRANE-PAVEMENT SYSTEMS[1]

This Standard is issued under the fixed designation D 3633; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

## 1. Scope

1.1 This method covers the measurement of the electrical resistivity of membrane-pavement systems when applied to concrete bridge decks.

1.2 Measurements shall be performed on the bituminous pavement surface covering the waterproofing membrane.

1.3 This method utilizes a measure of electrical resistance between the saturated top surface of the membrane and the reinforcing steel embedded in the concrete bridge deck.

1.4 The values measured represent the electrical resistance obtained with the equipment and procedures stated herein and do not necessarily agree or correlate with those using other equipment or procedures.

NOTE 1—The values stated in SI units are to be regarded as the standard.

## 2. Significance and Use

2.1 The method for measuring the electrical resistivity of membrane-pavement systems may be interpreted to indicate the effectiveness of such systems.

2.2 The method is predicated on the fact that an electrical connection between the surface of the pavement and the reinforcing steel in the concrete pavement cannot be made through an impermeable insulating membrane.

2.3 The method may be used for acceptance when the accepting agency specifies the minimum resistance value desired.

## 3. Apparatus

3.1 *Ohmmeter*, d-c, 20 000 Ω/V rating connected to a double-pole, double-throw switch box or a-c ohmmeter (switch box not required).

NOTE 2—When this method is used for acceptance, the accepting agency should specify the type of ohmmeter to be used.

3.2 *Insulated Wire*, No. 18, Belden test probewire or equivalent. Two spools, minimum 38 m (125 ft).

3.3 *Copper Plate*, 305 by 305 by 3.0 mm (12 by 12 by 1/8 in.), with the means for connecting the ohmmeter lead and a wooden handle approximately 1 m (39 in.) in length.

3.4 *Polyurethane Sponge*, 305 by 305 by 13 mm (12 by 12 by 1/2 in.), to be attached to the copper plate with rubber bands or other suitable means. When assembled this apparatus is called the probe.

3.5 *Pressure Spray Can*, 12-litre (3-gal) capacity, with a hose and spray nozzle.

## 4. Reagent

4.1 A wetting agent which, when added to the water, will break the surface tension and promote the penetration of the water through the bituminous pavement.

## 5. Sampling

5.1 Determine locations on the bridge deck to be tested by using either a grid pattern similar to that illustrated in Fig. 1 or by a random location system that will ensure that the bridge deck area to be tested will be adequately represented.

## 6. Procedure

6.1 Prepare the surface to be tested by removing all foreign material by sweeping or

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[1] This method is under the jurisdiction of ASTM Committee D-4 on Road and Paving Materials, and is the direct responsibility of Subcommittee D04.36 on Bridge Deck Protective Systems.

Current edition approved Nov. 25, 1977. Published January 1978.

scraping, or both. Do not use water to clean. The surface must be dry and clean before testing.

6.2 Uncoil an ample length of wire to reach all areas to be tested, attach the minus (−) jack of the ohmmeter to the reinforcing steel and the plus (+) jack to the 305 by 305 by 3.0-mm (12 by 12 by $^1/_8$-in.) copper plate of the probe. Then saturate the sponge with water containing the wetting agent.

Note 3—A direct connection from the ohmmeter to the top layer of reinforcing steel in the deck is desirable. However, if it is not practical to do this, the bridge railing, expansion joints, light standards, drainage scuppers, or other exposed steel, which is known to have contact with the top mat of reinforcing steel, may be used to provide this connection.

6.2.1 Check the ohmmeter battery for satisfactory charge, then zero the ohmmeter dial indicator.

6.2.2 Check for overall equipment operation and satisfactory circuit by placing the probe on exposed concrete deck curbing and observing the resistance reading on the ohmmeter. This reading will normally range from 1000 to 3000 Ω for various bridge decks. Place the probe at several locations along the curb and observe the readings, which must remain relatively constant as an indication of a complete electrical circuit and especially to ensure a good contact with the reinforcing steel.

Note 4—Excessive moisture in the pavement would cause invalid resistance readings. To give a measure of assurance against this, the resistance between any two test sites may be checked prior to the testing sequence described in 6.3 through 6.4. This is accomplished by attaching the ohmmeter to two probes, rather than a single probe, and the reinforcing steel. Immediate low readings (10 000 ± Ω) will indicate excessive moisture in the pavement and on top of the membrane and further testing of the entire deck, or at least such identified portions of it, should be postponed.

6.3 By means of the pressure spray can containing water mixed with 8 ml/litre (1 oz/gal) of wetting agent, wet a spot at each test location as determined in 5.1. Each spot should be large enough to accommodate the probe. Repeat the wetting of each test site three to five times in series. Take care that the wetted areas do not interconnect, either on the surface or at the pavement-membrane interface.

6.3.1 In order to be sure that the applied water has penetrated the pavement and is in contact with the membrane, select a representative, well-compacted, test location as a check point. After allowing ample time for moisture penetration at all test sites, place the probe on the check point and determine the resistance with the test apparatus assembled as described in 6.2. Repeat the wetting of all test sites followed by a determination of the resistance at the check point. Do this until the check point resistance reading has stabilized. The time required for the wetting process will vary, depending on the thickness and permeability of the pavement layer (30 min is normally sufficient).

Note 5—The reasoning behind the above requirement is that when the dense, very well-compacted pavement at the check point, which has been wetted equally along with all other test locations, achieves a stabilized resistance, then logically all other test sites should have stabilized also. It has been found expedient in some cases (due to high ambient temperatures) to place prewetted sponges at each test site after wetting in order to maintain saturation.

6.4 Proceed to test the bridge deck for electrical resistance. Place the probe at each test location and allow the ohmmeter reading to settle; note it. For a d-c ohmmeter, reverse the leads by tripping the double-pole switch. When the reading has settled again, note it and record the average of two such readings as the resistance for each point. (See Fig. 2 for an example of the recording procedure.)

Note 6—Reversal of the d-c meter leads by means of the switch box and averaging the readings reduces the error induced by galvanic coupling of the probe and the reinforcing steel. This is not necessary when using an a-c ohmmeter. If readings are taken using scales other than the highest range provided by the ohmmeter, note the scale used.

6.5 If it is desired to further define areas of low-resistance readings, establish a grid pattern with intersections at intermediate points not previously tested. These points should only be tested after allowing sufficient time for the moisture in the pavement to dissipate. Time for this will depend on the density and thickness of the pavement, as well as the ambient and pavement temperatures (usually a minimum of 24 h should be allowed).

## 7. Report

7.1 The resistance values shall be reported in a manner similar to that shown in Fig. 2. **Areas** that fail to meet any minimum requirements may be outlined on the grid sheet. If **desired**, these same areas may be outlined on **the bridge** deck. Any repairs or corrections **that** need to be made shall be noted on the **report** sheet.

## 8. Precision and Accuracy

8.1 Within the scope and significance of this method, a precision or accuracy statement has not been developed. A statement may be developed at a future date when more experience is accumulated.

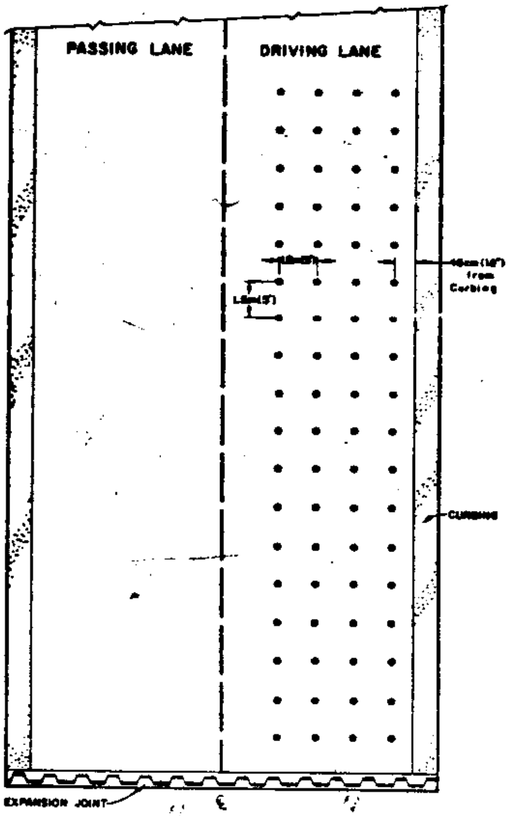

FIG. 1 Layout for Test Grid.

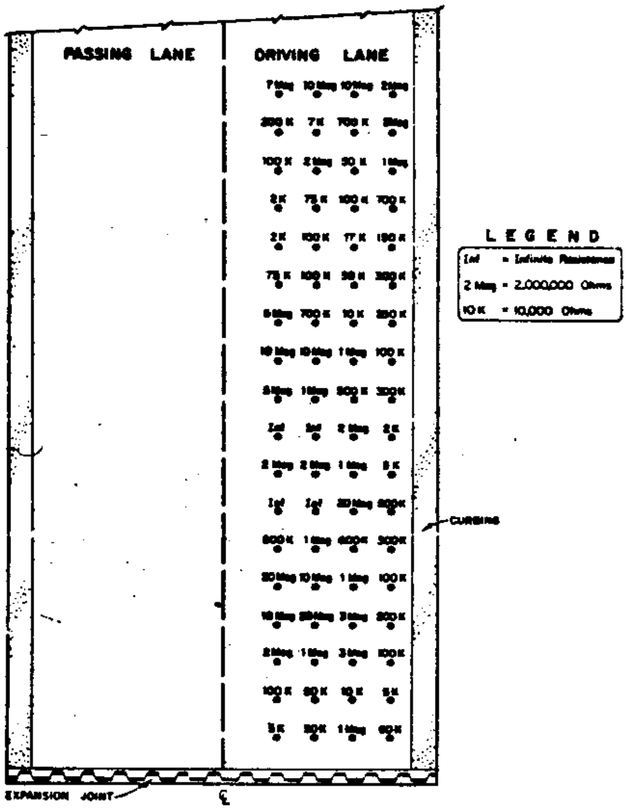

**FIG. 2 Typical Resistance Values.**

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## *Standard Method of Test for*

# Resistance of Concrete to Chloride Ion Penetration

## AASHTO Designation: T 259-78

### 1. SCOPE

**1.1** This method covers the determination of the resistance to the penetration of chloride ion of concrete, special concrete treatments or concrete overlays that will be subjected to traffic. It is intended for use in determining the effects of variations in the properties of concrete on the resistance of the concrete to chloride ion penetration. Variations in the concrete may include, but are not limited to, changes in the cement type and content, water-cement ratio, aggregate type and proportions, admixtures, treatments, curing and consolidation. This test method is not intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete.

### 2. TEST SPECIMENS

**2.1** The specimens for use in this test shall be slabs made and cured in accordance with the applicable requirements of AASHTO T 126, Making and Curing Concrete Test Specimens in the Laboratory.

Note 1—This method contemplates the use of a minimum of four specimens for each evaluation with each slab not less than 3 inches (76 mm) thick and 12 inches (305 mm) square.

**2.2** For this test the specimens shall be removed from moist curing at 14 days of age and stored for 14 days in a drying room of the type specified by AASHTO T 160, Length Change of Cement Mortar and Concrete.

**2.3** When the test method is used to evaluate concrete treatments, the slabs shall be fabricated from concrete having a cement factor of 658 lbs. (299 kg) per cubic yard (0.76 $m^3$) and a water-cement ratio by weight of 0.5. The treatments shall be applied in accordance with the manufacturer's recommendations.

**2.4** When a special overlay material is to be evaluated, the concrete slab shall be cast 2 inches (51 mm) thick using the mix design specified under paragraph 2.3 and then the special overlay material shall be placed 1 inch (25 mm) thick or to the manufacturer's recommended thickness and according to the manufacturer's recommended procedure.

### 3. PROCEDURE

**3.1** Immediately prior to the start of ponding with NaCl solution, 0.125 inch (3.2 mm) ± 0.0625 inch (1.6 mm) of the slab surface shall be abraded by some process such as grinding or sandblasting. No water shall be used in the abrading process.

**3.2** After abrasion 0.75 inch (19 mm) high by 0.50 inch (13 mm) wide dams should be placed around the top edge of all slabs except one, which will then become the control slab.

**3.3** All slabs shall then be returned to the drying room as specified under paragraph 2.2. The slabs with dams shall be subjected to continuous ponding with 3 percent sodium chloride solution to a depth of approximately 0.5 inch (13 mm) for 90 days. Glass plates shall be placed over the ponded solutions to retard evaporation of the solution. Placement of the glass plates shall not be done in such a manner that the surface of the slab is sealed from the surrounding atmosphere. Additional solution shall be added if necessary to maintain the 0.50 inch (13 mm) depth.

3.4 After 90 days of exposure the solution shall be removed from the slabs. The slabs shall be allowed to dry and then the surfaces shall be wire brushed until all salt crystal buildup is completely removed.

3.5 Samples for chloride ion analysis shall then be taken from all slabs in accordance with the rotary hammer procedure described in AASHTO T 260 or by dry coring (1.5 inch (38 mm) minimum diameter cores) and dry sawing. Three samples shall be obtained from each slab at each of the following depths:

1/16 inch (1.6 mm) to 0.5 inch (13 mm)
0.5 inch (13 mm) to 1.0 inch (25 mm)

The chloride content (parts per million by weight) of each sample thus obtained shall be determined in accordance with the procedures described in AASHTO T 260, Standard Method of Sampling and Testing for Total Chloride Ion in Concrete.

## 4. CALCULATIONS

4.1 The baseline chloride ion content for the test specimens shall be determined as the average chloride ion content of samples obtained from the 1/16 inch (1.6 mm) to 0.5 inch (13 mm) and 0.5 inch (13 mm) to 1.0 inch (25 mm) depths within the slab that was not ponded with 3 percent NaCl solution.

4.2 The absorbed chloride ion content of each sample from the ponded slabs shall be determined as the difference between the total chloride ion content of that sample and the baseline value calculated in section 4.1. If the result is less than zero, the result shall be reported as zero. The average chloride ion absorbed at each sampling depth shall be calculated.

## 5. REPORT

5.1 Reporting shall include: (1) each total chloride ion value determined in section 3.5, (2) the average and maximum baseline chloride ion (section 4.1), (3) each calculated absorbed chloride ion value determined in section 4.2, (4) the average and maximum absorbed chloride ion values calculated in section 4.2 for each depth.

# *Standard Method of Test for*

# Sampling and Testing for Total Chloride Ion in Concrete

## AASHTO Designation: T 260-78

## 1. SCOPE

1.1 This method covers a procedure for the determination of the total chloride ion content of aggregates, portland cement, mortar or concrete. The method is limited to materials that do not contain sulfides, but the extraction procedure, paragraphs 5.1 thru 5.6, may be used for all such materials.

## 2. APPARATUS

2.1 Samples may be obtained by one of two methods, 2.1.1 or 2.1.2.

2.1.1 Core drill

2.1.2 Rotary impact type drill with a depth indicator and drill or pulverizing bits of sufficient diameter to provide a representative sample of sufficient size for testing.

2.1.2.1 Sample containers capable of maintaining the sample in an uncontaminated state.

**2.1.2.2** Spoons of adequate size to collect the sample from the drilled holes.

**2.1.2.3** A "blow out" bulb or other suitable means of removing excess pulverized material from the hole prior to redrilling operations.

**2.1.2.4** A pachometer capable of determining the location and depth of steel reinforcement to ± 1/8 inch (±3 mm).

**2.2** Testing

**2.2.1** Chloride-ion or silver/sulfide ion selective electrode and manufacturer-recommended filling solutions.

**Note:** Suggested electodes are the Orion 96-17 Combination Chloride Electrode or the Orion 94-6 Silver/Sulfide Electrode or equivalents. The Silver/Sulfide electrode requires use of an appropriate reference electrode (Orion 90-02 or equivalent).

**2.2.2** A millivoltmeter compatible with the ion electrode.

**Note:** Suggested millivoltmeter is the Orion Model 701A Digital pH/$m_v$ meter or equivalent.

**2.2.3** Magnetic stirrer and teflon stirring bars.

**2.2.4** Burette with 0.1 ml graduations.

**2.2.5** Balance sensitive to 0.0001 gram with minimum capacity of 100 grams.

**2.2.6** Balance sensitive to 0.1 gram with minimum capacity of 1 kg.

**2.2.7** Hot plate, 250 to 400 C heating surface temperature.

**2.2.8** *Glassware* – 100 and 250 ml beakers, filter funnels, stirring rods, watch glasses, dropper; mortar and pestle; wash bottles.

**2.2.9** Sieve, U.S. Standard 50 mesh.

**2.2.10** Whatman No. 40 and No. 41 filter papers (or equivalent).

**Note:** If equivalent filter papers are used, they should be checked to confirm they do not contain chloride which will contaminate the sample.

## 3. REAGENTS

**3.1** Concentrated $HNO_3$ (sp. gr. 1.42).

**3.2** Sodium chloride, NaCl, reagent grade (primary standard).

**3.3** Standard 0.0100*N* NaCl solution. Dry reagent grade NaCl in an oven at 105 C. Cool, in a desiccator, weigh out 0.5844 grams, dissolve in distilled $H_2O$, and transfer to a 1-litre volumetric flask. Make up to the mark with distilled $H_2O$ and mix.

**3.4** Standard 0.01*N* $AgNO_3$. Weigh 1.7 grams of reagent grade $AgNO_3$, dissolve in distilled $H_2O$, filter into a 1-litre brown glass bottle, fill, and mix thoroughly. Standardize against 25.00 ml of the NaCl solution by the titration method given in paragraph 5.7.

**3.5** Distilled water.

**3.6** Methyl orange indicator.

**3.7** Ethyl alcohol, U.S.P. grade.

## 4. METHOD OF SAMPLING

**4.1** Determine the depth within the concrete for which the chloride content is desired. Use the pachometer to determine reinforcement bar location and depth.

**4.2** *Core Method* – Drill the core to chosen depth and retrieve.

**4.2.1** When samples are received in the laboratory in other than pulverized condition, the sample shall be crushed and ground to a powder. All sawing or crushing shall be done dry (i.e., without water). All material shall pass a number 50 mesh sieve. All pulverizing tools and sieves shall be washed with ethyl alcohol or distilled water and shall be dry before use with each separate sample (see note para. 4.3.7).

**4.3** Pulverizing Method

**4.3.1** Set the rotary hammer depth indicator so that it will drill to 1/2 inch (13 mm) above the desired depth.

**4.3.2** Using a drill or pulverizing bit, drill until the depth indicator seats itself on the concrete surface.

**4.3.3** Thoroughly clean the drilled hole and surrounding area utilizing the "blow out" bulb or other suitable means.

**4.3.4** Reset the depth indicator to permit 1/2 inch (13 mm) additional drilling.

**4.3.5** Pulverize the concrete until the depth indicator again seats itself on the concrete.

Note: Care must be exercised during this pulverizing operation to prevent the drill bit from abrading concrete from the sides of the hole above the sampling depth. To insure against this, some users utilize an 0.25 inch (6 mm) smaller diameter bit in this step than that used in para. 4.3.

**4.3.6** Collect at least 10 grams of the material remaining in the hole using a spoon and place in the sample container.

**4.3.7** If the sample, as collected, does not completely pass a 50 mesh screen, additional pulverizing shall be performed until the entire sample is finer than 50 mesh.

Note: During sample collection and pulverizing, personnel shall use caution to prevent contact of the sample with hands, or other sources of body perspiration or contamination. Further , all sampling tools (drill bits, spoons, bottles, sieves, etc.) shall be washed with ethyl alcohol or distilled water and shall be dry prior to use on each separate sample. Ethyl alcohol is normally preferred for washing because of the rapid drying which naturally occurs.

## 5. PROCEDURE

**5.1** Weigh to the nearest milligram a 3 gram powdered sample representative of the material under test.

Note: Some users dry the sample to constant weight in a 105 C oven and determine the dry sample weight prior to analysis. This optional procedure provides a constant base for comparison of all results by eliminating moisture content as a variable. It is generally believed that drying is only necessary when very high accuracy is desired (see Reference 1 for data in this area).

Transfer the sample quantitatively to a mortar; add 10 ml of hot (90 to 100 C) distilled $H_2O$ to the mortar, swirling to bring the powder into suspension. Carefully grind the slurry with a pestle until all lumps are gone. Very little grinding will be necessary for soft aggregates, but considerable effort will be required for samples containing hard aggregates.

Note: Sample particle size after grinding should be such that it will pass a 100 mesh screen. Further, about 75 percent of a properly ground sample will pass a 200 mesh screen. It is suggested that the analyst grind several trial samples, in accordance with the above procedure and then dry the samples and determine the particle size as a means of defining the grinding required for actual samples.

**5.2** Transfer the slurry quantitatively from the mortar through a funnel into a 100 ml beaker, rinsing the funnel lightly with hot distilled $H_2O$. Add 3 ml concentrated $HNO_3$ to the mortar and stir with the pestle to completely dissolve any cement left in the mortar. Transfer the contents of the mortar through the funnel while continuously stirring the beaker with a glass stirring rod. Rinse the mortar, pestle, inside of the funnel and the tip of the funnel with hot distilled $H_2O$.

Note: Too rapid transfer of the acid into the 100 ml beaker will cause excessive foaming or frothing of samples with calcareous aggregates or organic components and resultant risk of sample loss.

**5.3** Make up the solution in the 100 ml beaker to approximately 50 ml with hot distilled $H_2O$. Stir thoroughly to ensure complete sample digestion. Add five drops of methyl orange indicator and stir. If yellow to yellow-orange color appears, solution is not sufficiently acidic. Add additional concentrated $HNO_3$ dropwise with continuous stirring until a faint pink or red color persists in the solution. Cover with a watch glass, retaining the stirring rod in the beaker.

Note: Due to the presence of relatively insoluble materials in the sample, the solution generally will have a strong gray color, making the detection of the indicator color difficult at times. Running of several trial samples is suggested to give the analyst practice in detecting the indicator color.

**5.4** Bring the solution in the covered 100 ml beaker to a boil on a medium heat (250 to 400 C) hot plate, and then boil for a full minute with care to avoid frothing and spillovers. Remove from heat.

Note: The analysis can be stopped at this point and the sample allowed to cool in an HCl fume-free area if it is necessary. Before proceeding to the next step, however, the solution must again be brought to a boil.

**5.5** Prepare a funnel fitted with double filter paper (Whatman No. 41 over No. 40 filter paper or equivalents) and a 250 ml beaker to receive the filtrate. Carefully lift the watch glass from the 100 ml beaker, without tilting it, and wash any adhering drops into the filter paper with hot distilled water. Then filter the hot solution into the 250 ml beaker. Proceed carefully, employing the stirring rod to aid quantitative transfer of the solution into the filter funnel. Wash the inside of the 100 ml beaker and the stirring rod twice with hot distilled $H_2O$. Transfer the washings through the filter into the 250 ml beaker. Finally, carefully wash the outside of the pouring lip of the 100 ml beaker with hot distilled $H_2O$ into the filter.

**5.6** Wash the filter paper five to ten times with hot distilled $H_2O$, being careful not to lift the paper away from the funnel surface. Finally, lift the filter paper carefully from the funnel and wash the outside surface of the paper with hot distilled $H_2O$; then wash the tip of the funnel. The final volume of the filtered solution should be 125 to 150 ml. Cover with a watch glass and allow to cool to room temperature in an HCl fume-free atmosphere.

**5.7** Two alternate methods are available to determine the $Cl^-$ content of the solution. Both methods utilize an ion selective electrode ($Cl^-$ or $Ag^+$) and both methods for the purpose of this analysis give results of essentially equal accuracy and precision. However, Method II offers a substantial decrease in time required for analysis over Method I.

**5.7.1** Alternate Method I: Potentiometric Titration

Fill the $Cl^-$ or the $Ag^+$ electrode with the solution(s) recommended by the manufacturer, plug it into the millivoltmeter (preferably the type with a digital rather than a dial readout), and determine the approximate equilvalence point by immersing the electrode in a beaker of distilled $H_2O$. Note the approximate millivoltmeter reading (which may be unsteady in $H_2O$).

Take the cooled sample beaker from 5.6 and carefully add 4.00 ml of 0.01*N* NaCl, swirling gently. Remove the beaker of distilled $H_2O$ from the electrode, wipe the electrode with absorbent paper, and immerse the electrode in the sample solution. Place the entire beaker-electrode assembly on a magnetic stirrer and begin gentle stirring.

Using a calibrated buret, add gradually and record the amount of standard 0.01*N* $AgNO_3$ solution necessary to bring the millivoltmeter reading to −40 mv of the equivalence point determined in distilled $H_2O$. Then add standard 0.01*N* $AgNO_3$ solution in 0.10 ml increments recording the millivoltmeter reading after each addition.

As the equivalence point is approached, the equal additions of $AgNO_3$ solution will cause larger and larger changes in the millivoltmeter reading. Past the equivalence point, the changes per unit volume will again decrease. Continue the titration until the millivoltmeter reading is at least 40 mv past the approximate equivalence point.

The endpoint of the titration usually is near the approximate equivalence point in distilled water and may be determined by (1) plotting the volume of $AgNO_3$ solution added versus the millivoltmeter readings. The endpoint will correspond to the point of inflection of the resultant smooth curve, or (2) calculating the differences in millivoltmeter readings between successive $AgNO_3$ additions and calculating the total volume of $AgNO_3$ which corresponds with each difference (i.e., the midpoints between successive additions).

**Example:**

| Raw Data | | Differences | |
|---|---|---|---|
| **Titrant Volume** | **Millivolt Reading** | **Titrant Midpoints** | **Millivolt Differences** |
| 4.2 ml | 130.0 | 4.25 ml | 5.0 |
| 4.3 ml | 135.0 | 4.35 ml | 7.0 |
| 4.4 ml | 142.0 | 4.45 ml | 10.0 |
| 4.5 ml | 152.0 | etc. | |
| etc. | | | |

The endpoint will be near the midpoint which produced the largest change in millivoltmeter reading. It may be determined by plotting midpoints versus differences and defining the $AgNO_3$ volume which corresponds to the maximum difference on a smooth, symetrical curve drawn through the points. However, it can usually be estimated accurately without plotting the curve by choosing the midpoint which corresponds to the maximum difference and adjusting for asymmetry, if any. In other words, if the differences on each side of the largest difference are not symmetrical, adjust the endpoint mathematically in the direction of the larger differences. Detailed examples of this adjustment are contained in Reference 1.

**5.7.2** Alternate Method II: Gran Plot Method

This method is compatible with either a $Cl^-$ or $Ag^+$ ion-selective electrode. Attach the electrode of choice to a compatible digital millivoltmeter after filling with required solutions as per the electrode manufacturer's instructions. Clean the electrode with distilled $H_2O$ and pat dry with absorbent paper.

Weigh the cooled sample and beaker from 5.6 without the watch glass and record the weight. Using a calibrated buret, titrate the sample to 225 mv ± 5 mv ($Cl^-$ electrode) or 310 ± 5 mv ($Ag^+$ electrode) with standard 0.01*N* $AgNO_3$ solution. Record the volume added and the millivoltmeter reading.

Continue to titrate in 0.50 ml increments recording the volume added and the millivoltmeter reading for each increment. Add and record the data for at least five increments. Empty, clean, dry and weigh the beaker. Subtract beaker weight from beaker + solution weight determined above to define solution weight.

Additional information on the Gran Method is given in Reference 2.

## 6. CALCULATIONS

**6.1** Alternate Method I—Potentiometric Titration

Determine the endpoint of the titration as described in 5.7.1 by either plotting a curve or estimating from the numerical data. Calculte the percent $Cl^-$ ion from the equation:

$$\%Cl^- = \frac{3.5453\ (V_1N_1\text{-}V_2N_2)}{W}$$

Where $V_1$ = endpoint in ml $V_2$ = Volume of NaCl solution added, in ml
$N_1$ = normality of $AgNO_3$ $N_2$ = Normality of NaCl solution
W = Weight of original concrete sample in grams

**6.2** Alternate Method II—Gran Plot Method

Calculate corrected values for each of the volumes recorded in 5.7.2 by the equation:

$$V_{correct} = \frac{V_{record}}{W/100}$$

Where W = original solution weight in grams
$V_{record}$ = Volumes recorded in ml

If any of the V correct values are greater than 10, see para. 6.3. If less than 10, plot these corrected values versus the corresponding millivolt readings on Orion Gran Plot Paper (10 percent volume corrected type with each major vertical scale diversion equal to 5 millivolts) or equivalent. Draw the best straight line through the points and read the endpoint at the intersection of the line with the horizontal axis of the graph. Calculate the actual endpoint by the equation:

$$E_a\text{, ACTUAL ENDPOINT} = E_g\left(\frac{W}{100}\right)$$

where $E_g$ = Endpoint determined from graph in ml
W = Weight of solution in grams

Then $\% Cl = \dfrac{3.5453\ E_aN}{Wc}$

Where $E_a$ = Actual endpoint, in ml
N = Normality of $AgNO_3$ solution

**6.3** Supplementary Gran Method Calculations:

When the V correct volumes determined in 6.2 are greater than 10, discard the values and follow the following procedure.

Choose a constant which, when subtracted from all V record volumes, yields values less than 10 ml.

Note: This constant, designated as X in the formulas below, is normally assiged an even value such as 5, 10, 15, 20, etc.

Calculate a revised solution weight $W_r$ as

$$W_r = W + X$$

where W = original solution weight in grams
X = the constant

Then calculate corrected volumes for each recorded volume as:

$$V_{correct} = \frac{V_{record} - X}{W_r/100}$$

Plot these values and determine the graph endpoint $E_g$ as described in para. 6.2. The actual endpoint, $E_a$ is then:

$$E_a = E_g\left(\frac{W_r}{100}\right) + X$$

where $E_a$ = actual endpoint in ml
$E_g$ = endpoint from graph in ml
$W_r$ = revised solution weight in grams
$X$ = the constant chosen above.

Calculate the chloride content using the formula given in para. 6.2.

**6.4** The percent chloride may be converted to pounds to $Cl^-$ per cubic yard of concrete as follows:

$$\text{lbs } Cl^-/yd^3 = \%\ Cl\left(\frac{UW}{100}\right)$$

**Where**

**UW = Unit weight of concrete per cubic yard.** (metre)

**Note:** A unit weight of 3915 lbs/yd³ is often assumed for normal structural weight concrete when the actual unit weight is unknown.

## 7. REFERENCES

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# APPENDIX C.

# FIELD TRIP REPORT

# FIELD TRIP REPORT

As part of this study, the project staff were to familiarize themselves with the range of bridge deck deterioration existing in the State system. As a result of this brief examination of several bridges, both the project staff and the WSDOT staff were able to "calibrate" the various distress modes in bridge decks by visually observing deterioration of concrete bridge decks caused by deicing chemicals.

On Thursday, June 17, 1982, Dr. Ronald L. Terrel, project investigator, and Mr. Khossrow Babaei, co-principal investigator, from TRAC, accompanied by Mr. Tom Roper, Senior Associate Bridge Engineer, and Mr. Pat Morin, Bridge Engineer, from WSDOT, visited the following deteriorated bridge decks in the Seattle area:

1. Ship Canal (I-5)
2. 145th Street Overpass (I-5)
3. North Everett (Snohomish River) (I-5)
4. Woodinville Interchange (I-5, I-405)

Spalling and cracking of deck concrete was noted in all installations. Most of the spalls were already patched with Set-45, which is a fast-curing patching material. The spalls and cracks could be seen mainly in wheel paths and around the columns where negative moment exists. The reason for this is that the salt can easily wash into the cracks which are over the topmost reinforcing bars and cause corrosion of the reinforcement. The stress caused by the corrosion and the pressure from passing traffic then creates the spalls.

These bridge decks were designed and built to specifications which have subsequently proven to be inadequate to prevent the corrosion of reinforcement. The bridge decks all had small depths of cover (about ¼ in) and high water/cement ratios (about 0.45), which are the main causes of salt intrusion.

ACP overlays and waterproofing membranes will be applied on bridge nos. 1, 3 and 4 as the protective system, after removal and replacement of deteriorated and contaminated concrete. An impressed current cathodic protection might be installed on bridge no. 2, since the concrete is not deteriorated and AC power supply is already available.

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